NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

# NCHRP Report 380

# Transverse Cracking in Newly Constructed Bridge Decks

Transportation Research Board
National Research Council

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# Transverse Cracking in Newly Constructed Bridge Decks

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## NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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# FOREWORD

*By Staff*
*Transportation Research Board*

This report contains the findings of a study that was performed to establish the factors and combinations of factors pertaining to concrete materials, design details, and construction practices that influence the occurrence of early transverse cracking in bridge decks. The report provides a comprehensive description of the research, including Guidelines to help highway engineers select the concrete materials and construction practices that should reduce or eliminate the occurrence of transverse cracking in newly constructed bridge decks. Also, the report describes a recommended test method for evaluating the cracking tendency of concrete. The contents of this report will be of immediate interest to bridge engineers, researchers, and others concerned with the design and construction of bridge structures.

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Many concrete bridge decks develop transverse cracking shortly after construction. These cracks accelerate corrosion of reinforcing steel and lead to concrete deterioration, damage to components beneath the deck, and unsightly appearance. These cracks shorten the service life and increase maintenance costs of bridge structures.

Under NCHRP Project 12-37, "Transverse Cracking in Newly Constructed Bridge Decks," Wiss, Janney, Elstner Associates, Inc., was assigned the tasks of determining the major factors or combinations of factors that contribute to transverse deck cracking in newly constructed bridge decks, and recommending Guidelines for preventing or reducing the occurrence of such cracking. To accomplish these objectives, the researchers reviewed relevant domestic and foreign literature, surveyed all U.S. departments of transportation and several transportation agencies overseas, performed analytical studies and laboratory tests, and conducted field measurements on a bridge structure during and shortly after deck construction. The report documents the work performed under NCHRP Project 12-37 and describes the relevance of concrete materials, design details, and construction practices to the occurrence of early deck cracking.

The recommended Guidelines, summarized in this report, identify and rank the factors and combinations of factors pertaining to concrete materials, design details, and construction practices that influence the occurrence of transverse cracking in newly constructed bridge decks. The Guidelines outline recommended actions to help reduce or eliminate early deck cracking.

The proposed cracking-tendency test procedure, described in this report, can be used to compare the resistance of different concretes to early cracking. The test procedure will be particularly useful to highway agencies and is recommended for consideration and adoption by AASHTO as a standard test method.

# CONTENTS

**ACKNOWLEDGMENTS**

The research reported herein was performed under NCHRP Project 12-37 by Wiss, Janney, Elstner Associates, Inc. (WJE). Paul D. Krauss, P.E., and Ernest A. Rogalla, S.E., were Co-Principal Investigators. Matthew R. Sherman performed the majority of the cracking-tendency tests, and David B. McDonald performed the body of the literature review. Andrew E. N. Osborn directed the field studies. This work was done under the general supervision of Donald W. Pfeifer, Principal and Vice President of WJE.

# TRANSVERSE CRACKING IN NEWLY CONSTRUCTED BRIDGE DECKS


## SUMMARY

Many concrete bridge decks develop transverse cracks soon after construction. These cracks are typically full-depth and spaced 1 to 3 m (3 to 10 ft) apart along the length of the bridge. Transverse cracks may shorten the service life of a bridge and increase maintenance costs. They can accelerate corrosion of reinforcing steel, deteriorate concrete, damage components beneath the deck, and damage appearance.

Researchers and transportation agencies do not agree on a minimum crack width that affects performance. Typical acceptable crack widths for structures subject to deicing chemicals range from near 0 to 0.2 mm (0.008 in.). Denmark, Japan, and Switzerland typically specify a maximum crack width of 0.2 mm (0.008 in.) on conventionally reinforced decks. Only two U.S. transportation agencies limit crack widths; one limits crack width to 0.18 mm (0.007 in.), and the other specifies less than 15.2 m (50 ft) of cracks wider than 0.5 mm (0.020 in.) per 46.5 $m^2$ (500 $ft^2$) of deck surface area. The orientation and shape of cracks with respect to the deck reinforcement significantly affect deterioration. A conservative approach of bonding or sealing all visible cracks is suggested when attempting to achieve the most durable structure in an aggressive environment.

A survey was sent to all U.S. departments of transportation (DOTs) and several transportation agencies overseas to learn the extent of early transverse cracking. Fifty-two agencies from the United States and Canada responded to the survey. The respondents estimate that more than 100,000 bridges in the United States, about half the bridges monitored by the respondents, developed early transverse cracking. Sixty-two percent of the agencies consider early transverse cracking a problem, 24 percent do not consider this a problem, and 14 percent have no opinion. Many agencies that do not consider transverse cracking a problem nonetheless reported extensive cracking. Fifteen percent believed that all of their decks have transverse cracks.

This project studied transverse cracking with theoretical analyses, field instrumentation, and laboratory research. The elastic equations derived to calculate thermal and shrinkage stresses in a bridge deck are presented in Appendix F, which is not published herein. The results of the monitoring of a bridge deck replacement were compared to theoretical behavior calculated by using the equations derived for this project. Laboratory studies of the cracking tendency of various mix and environmental parameters were also performed.

The analytical studies used conventional and finite element analysis techniques to evaluate the influence of various factors on deck tensile stresses and cracking. These factors included concrete drying shrinkage, creep, hydration temperatures and other thermal effects, position and amount of reinforcing steel, girder size and spacing, single- and multispan conditions, age, and other parameters. This parameter study analyzed bridges with steel girders; reinforced concrete girders; precast, prestressed concrete girders; and cast-in-place, post-tensioned girders. Simple-span and two-span structures were studied. Three different temperature profiles and two different deck drying shrinkage profile conditions were studied. Approximately 18,000 bridge system scenarios were analyzed. These analytical techniques enabled a more thorough investigation of the parameters that affect cracking than field work or laboratory testing alone would have.

This project identified and ranked the factors or combinations of factors that contribute to transverse cracking of newly constructed bridge decks. These analyses determined that, for most bridges, concrete properties affect deck cracking more than any other factors. As a result, a test procedure was developed to measure the cracking tendency of different concretes. In brief, the test involves casting a concrete ring against a steel inner ring that restrains the shrinking concrete and usually causes cracking. This test procedure is proposed for adoption by AASHTO so that transportation agencies can test and develop concrete mixes that resist transverse cracking.

The major design factors affecting cracking are span type, concrete strength, and girder type. Moderate factors included dead-load deflections, base restraint, concrete cover, span length, and quantity of reinforcement.

For most bridges, the girders significantly restrain the deck against its natural shrinkage and thermal movement, causing stresses in the deck. These stresses and the risk of transverse deck cracking are caused mainly by the amount of restraint provided to the deck by the girders, differential shrinkage and thermal movements of the deck and girders, concrete effective modulus of elasticity, and shrinkage and thermal strains. Material properties such as cement content, cement composition, early age elastic modulus, creep, aggregate type, concrete temperature during placement, heat generated during hydration, and drying shrinkage also influence cracking.

Many of the transportation agencies surveyed stated that concretes with 335 kg/m$^3$ (564 lb/yd$^3$) of cement cracked less than those with 390 kg/m$^3$ (658 lb/yd$^3$) of cement. These agencies generally recommended that the water-cement ratio be reduced to between 0.41 and 0.49, and that peak concrete temperatures be lowered to reduce stresses during cooling. Many agencies recommended aggregates that have low shrinkage values and Type II cement to reduce shrinkage and temperature during early hydration.

The survey respondents recommended stiffer decks and additional reinforcement to reduce transverse deck cracking. They also recommended a minimum concrete cover of 38 mm (1.5 in.) over the reinforcement, and a maximum cover of 75 mm (3 in.). To reduce cracking, the minimum recommended thickness of the decks should be 200 to 230 mm (8 to 9 in.). Decreased spacing and size of temperature and shrinkage steel were recommended. This steel should be placed on top of the primary slab reinforcement.

The transportation agencies generally believed that construction practice can affect transverse cracking. Curing is the most important aspect of construction, and effective curing should commence as soon as possible. They suggested wind breaks and sun shades during periods of high evaporation to reduce drying. Nighttime concrete construction is preferred because of lower air temperatures, lower solar radiation, and higher relative humidity.

Weather conditions during concrete placement greatly influence the number of medium and large cracks that develop early in the life of the structure. More cracking was observed when concrete was placed during low humidities and high evaporation rates. Cracking also may be worse when concrete is cast at low temperatures or high temperatures. The transportation agencies cited vibration and finishing procedures as potential factors that cause early cracking. Pour length and combination of pours did not seem to be a major influence on the amount of transverse cracking. Traffic-induced vibrations during hardening were also found not to be detrimental to deck concrete.

CHAPTER 1

# INTRODUCTION AND RESEARCH APPROACH

## PROBLEM STATEMENT

Concrete bridge deck cracking is a prevalent problem in the United States. Transverse cracking occurs in most geographical locations and climates, and in many types of bridge superstructures. The objective of this research was to determine the major factors or combination of factors that contribute to early transverse deck cracking. The research was limited to conventionally reinforced bridge decks. Deck cracking can be caused by plastic shrinkage, shrinkage stresses, thermal stresses, bending stresses, fatigue, corrosion of embedded steel, and chemical reactions. This project identified construction methods, concrete materials, and structural design procedures that reduce or eliminate transverse deck cracking.

Surveys were sent to all U.S. DOTs and several transportation agencies overseas to learn the extent of deck cracking. Fifty-two agencies within the United States and Canada responded to the survey. Sixty-two percent considered transverse cracking at early ages to be a problem, 24 percent did not consider early transverse cracking a problem, and 14 percent had no opinion. Many that did not consider transverse cracking to be a problem nonetheless reported extensive cracking.

This project was limited to early transverse cracking of bridge decks. These cracks typically are full-depth cracks, aligned above the transverse reinforcing. The surface crack widths typically ranged from 0.05 mm (0.002 in.) to 0.65 mm (0.025 in.), and the cracks were spaced usually 1 to 3 m (3 to 10 ft) apart. These types of cracks are common, and are often mistakenly called drying shrinkage cracks. Shrinkage is not synonymous with cracking.

The causes and prevention of plastic shrinkage cracking are well understood. Plastic shrinkage cracking occurs when the evaporation rate exceeds the bleed rate of the plastic concrete. Plastic cracks typically are very wide at the surface, but narrow substantially with depth. They rarely exceed 50 to 75 mm (2 to 3 in.) in depth. Plastic cracks are short and can occur in any direction. Plastic cracking can be prevented by using monomolecular evaporation-retarding films, water fogging, wind breaks, or casting concrete when evaporation is low. Plastic shrinkage cracking is not the focus of this project. Also, this project did not examine cracking at later ages caused by reactive aggregates or corrosion of embedded steel.

### Crack Development

Concrete bridge decks develop transverse cracks when longitudinal tensile stresses in the deck exceed the tensile strength of the concrete. These tensile stresses are caused by temperature changes in the concrete, concrete shrinkage, and sometimes bending from self-weight and traffic loads. A combination of shrinkage and thermal stresses causes most transverse bridge deck cracking.

Shrinkage and temperature stresses develop in all bridge decks, because the girders restrain the natural thermal and shrinkage movement of the deck. When the deck and girders consist of different materials (steel and concrete, or different concretes) with different thermal expansion rates, even a constant temperature change will cause stresses because the different materials expand differently and cannot expand where they are attached. Temperatures in a bridge are rarely uniform or linearly distributed, and shrinkage is also nonlinearly distributed. Nonlinear shrinkage and temperature changes cause stress, even without restraint.

Many factors affect shrinkage and thermal stresses. For example, geographic location affects these stresses, because environment affects early hydration temperatures, drying shrinkage, and final temperature cycling. Material properties and bridge geometry also affect shrinkage and thermal stresses.

Unrestrained concrete expands when it is heated, contracts when it is cooled, and shrinks as it dries. These thermal and shrinkage movements are expressed in terms of strain. Strain by itself does not necessarily cause stress (necessary for cracking). When concrete undergoes a uniform or linearly distributed shrinkage or temperature change, it will not develop stresses if it is not physically restrained against movement. However, if restrained, the force or pressure restraining the concrete causes stress.

Bridge elements restrain thermal and shrinkage strains in a deck, causing stress. Because there is considerable friction between the deck and its supporting girders, the girders restrain deck strains when they do not have temperature or shrinkage strains identical to the deck. This restraint is usually worst with steel girders, because steel girders do not shrink and steel usually has a different coefficient of thermal expansion. To a lesser extent, embedded reinforcement in the deck also restrains the deck against shrinkage and against thermal movements when the steel has a different coefficient of thermal expansion than the concrete.

Restraint can cause large shrinkage and thermal stresses. For example, if the concrete has a free-shrinkage of 500 microstrain ($\mu\epsilon$), but it is restrained and allowed to shorten only 250 $\mu\epsilon$, the restraint is 50 percent. A concrete with a modulus of elasticity of 28 GPa ($4 \times 10^6$ psi) might have an effective modulus of only 14 GPa ($2 \times 10^6$ psi), because of creep. The resultant tensile stress would be the product of the strain (500 $\mu\epsilon$) times the restraint (50 percent) times the effective modulus of elasticity [14 GPa ($2 \times 10^6$ psi)] for a resultant tensile stress of 3.4 MPa (500 psi). If the tensile strength of the concrete is greater than 3.4 MPa (500 psi), cracking will not occur. However, additional tensile stresses from thermal gradients or loading could crack such a concrete. Therefore, effects of shrinkage and temperature changes, effective concrete modulus, restraint conditions, tensile strength, and loading conditions must be considered. Several examples follow.

If a deck could be separated from its girders, the three basic temperature changes would produce the movements (free-strains) shown in Figure 1. When both the girders and deck have similar temperatures increases (Figure 1, Condition 1), both expand similarly; such a temperature change may occur between uniform summer and winter conditions. Sustained solar radiation on a bridge deck, early exothermic cement hydration of the concrete deck, or both may cause a nearly uniform temperature change in the deck while the temperature changes in the girders remain small (Figure 1, Condition 2); this can cause a large free-strain difference between the deck and girders. Because solar radiation heats the upper surface of a deck quicker than the heat conducts to lower depths, a nearly linear temperature gradient often occurs in bridge decks during the morning (Figure 1, Condition 3); such a condition would cause the free deck to curve convex upward. A nearly linear temperature gradient can also occur during early evening or during rains as the upper surface of the warmed deck radiates heat and cools quicker than lower depths. Temperatures in a bridge are constantly changing, but they can often be represented by one of these three distributions or by combinations of these three.

Because a bridge deck is typically composite with its girders (even if it is not intentionally designed to be so), the free-strain or curvature differences shown in Figure 1 cannot

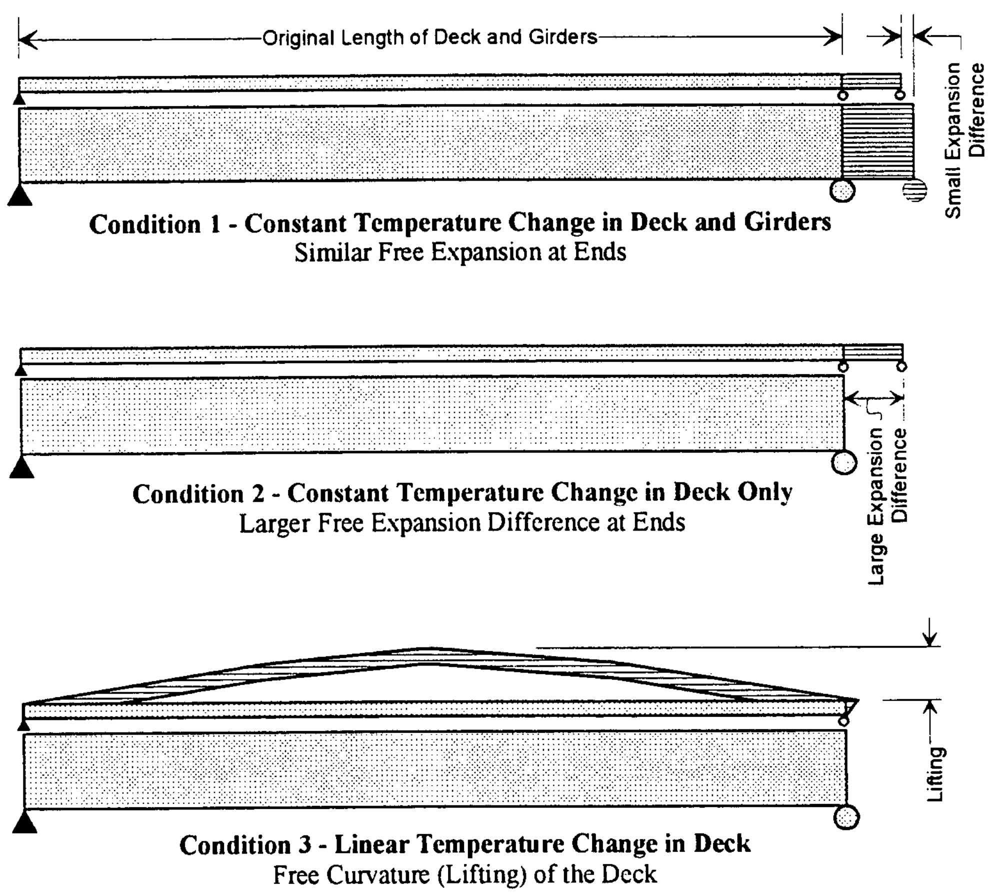


*Figure 1. Strain effects of various temperature changes.*

occur. Instead, normal and shear stresses develop at the interface between the deck and girders so that there is compatibility (identical strain and curvature) at the interface. For analysis, a 30°C (54°F) maximum temperature decrease was applied to a representative steel-girder composite bridge for the three conditions illustrated in Figure 1. Figure 2 shows the resulting stresses in the concrete deck and steel girder. The positive values indicate tensile stresses.

As Figure 2 indicates, each temperature decrease produces drastically different stresses. A uniform temperature change in both the deck and girders for Condition 1 causes small deck stresses less than 41 kPa (6 psi) because little free-strain incompatibility (differential movement) exists at the interface. Large, nearly uniform tensile stresses of about 2.2 MPa (310 psi) can develop in the deck from a uniform temperature change only in the deck as shown in Condition 2. Even larger tensile stresses of 5.9 MPa (860 psi) can occur from a nearly linear temperature change only in the deck, with tensile stresses on one surface of the slab and compressive stresses on the opposite surface for Condition 3. For Condition 3, the unrestrained bowing potential for a deck 30 m (98.4 ft) long would be 160 mm (6.3 in.); whereas, for Condition 2, the unrestrained movement of the deck would be 8.9 mm (0.35 in.), and for Condition 1, the difference in unrestrained girder and deck movements would be 1.6 mm (0.06 in.).

Shrinkage and thermal stresses in a bridge deck depend on the restraint provided to the deck by the girders. Because girders restrain a deck at the soffit and not the centroid of the deck, the eccentric restraint causes both membrane (in-plane) and bending stresses in the deck. These effects are complicated, and stress reversals within the deck can occur when bending stresses exceed membrane stresses. Even uniform shrinkage or temperature changes can cause tensile stresses on one surface of the deck and compressive stresses on the opposite face.

When a uniform free-strain is applied to a deck (Figures 1 and 2, Condition 2), the girders partially restrain longitudinal movement of deck, and the eccentric restraint causes the deck to bend. The longitudinal restraint causes membrane stresses in the deck, and the bending causes bending stresses. For some geometries, bending stresses are small,

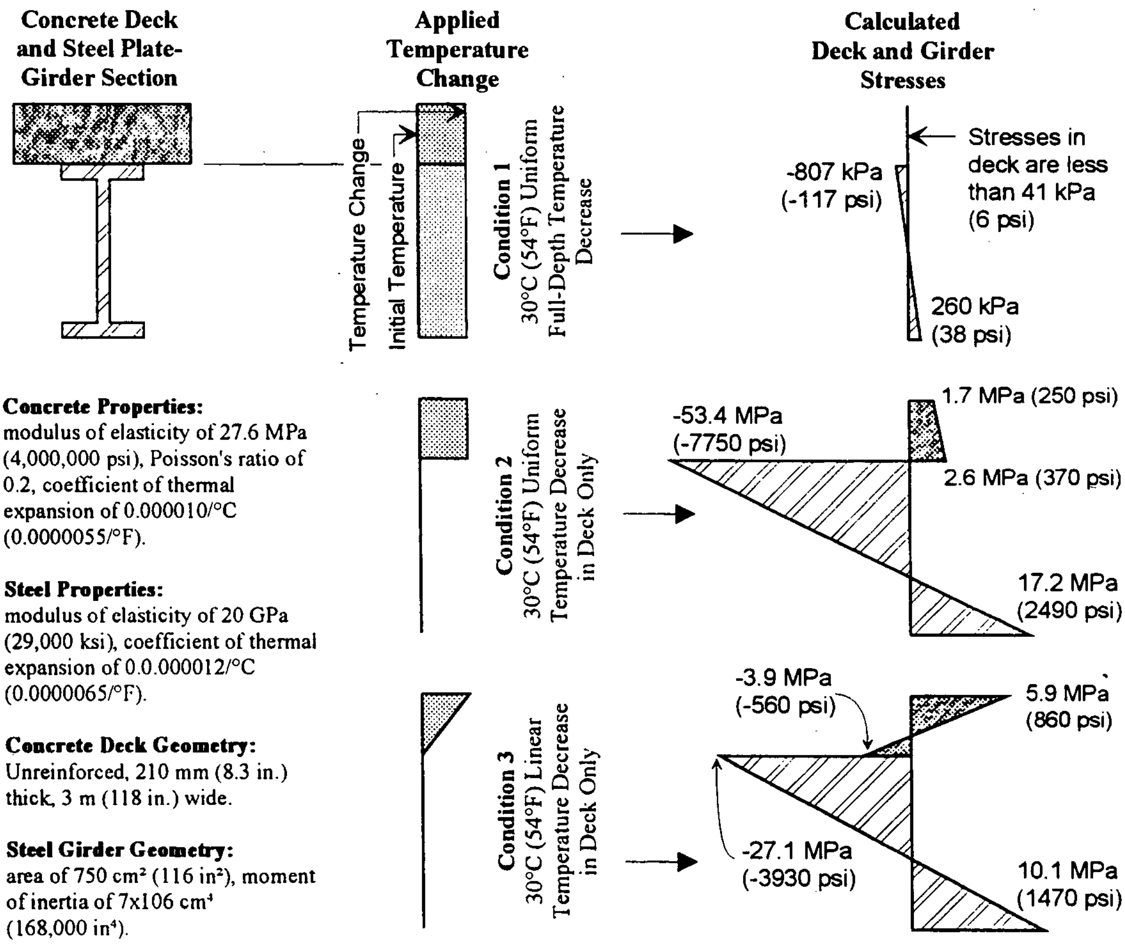


*Figure 2. Example deck and steel girder stresses for various temperature changes.*

and restraint stresses in the deck are nearly uniform. For other geometries, bending stresses are larger than membrane stresses, and stress reversal develops in the deck. When large steel plate girders or large concrete girders support the deck, they restrain approximately 15 to 20 percent of the uniform free-strain at the upper surface of the deck and approximately 25 to 35 percent at the soffit. Smaller steel or concrete girders typically restrain little or no free-strain at the upper surface and between 20 and 25 percent at the soffit.

When applied free-strains in the deck are linear (Figures 1 and 2, Condition 3), girders restrain most of the curvature that would develop in the deck if the deck was separated from the girders. Steel girders typically restrain between 85 and 95 percent of this curvature, whereas concrete girders usually restrain between 75 and 95 percent; larger girders restrain more curvature. For the example shown in Figure 2, Condition 3, a composite span 30 m (98.4 ft) long would bow 8 mm (0.32 in.), only 5 percent of the 160 mm (6.3 in.) bowing potential. However, because of an interface shear, final strains in the deck are less than the curvature restraint suggests. Large steel or concrete girders can restrain about 60 percent of the free-strain at the upper surface, and smaller girders often restrain between 35 and 45 percent at the upper surface. When linear free-strains are applied to most decks, bending stresses are larger than membrane stresses, and stress reversals develop in the deck.

### Bridge Deck Durability

Understanding why cracking is a problem is important. Transverse cracking can cause accelerated corrosion of reinforcing steel, deterioration of concrete, leakage onto structural members and components beneath the deck, and poor appearance. Only two state DOTs currently require repair of cracks in new bridge decks.

Wide cracks may be aesthetically unacceptable. Crack width, lighting conditions, viewing distance, and crack length affect the appearance of a crack. In some past cases, a crack width of 0.3 mm (0.010 in.) was regarded as the largest aesthetically acceptable width.

When concrete cracks in a corrosive environment, embedded reinforcing steel can rapidly corrode. Corrosive chloride ions and carbon dioxide penetrate through the cracks into the concrete and to the reinforcing steel. There are many contrary opinions relating crack width to corrosion. Some research studies found a correlation between crack width and concrete deterioration, but others did not. The following maximum crack widths are often recommended:

- 0.1 mm (0.004 in.) for severe exposure to deicing chemicals or for water tightness,
- 0.2 mm (0.008 in.) for normal exterior exposures or interior exposures subjected to high humidities, and
- 0.3 mm (0.012 in.) for internal protected structures.

A very wide surface crack that quickly narrows with depth may not be as detrimental as a narrower surface crack with parallel sides. During recent investigations on cracked decks, the authors discovered water leakage through cracks with surface widths of only 0.05 to 0.20 mm (0.002 to 0.008 in.). The crack width at the bar depends on the origin of the crack, amount of cover, stress in the steel, concrete creep, reinforcement ratio, arrangement of the bars, bar diameter, and stress profile in the deck.

The orientation of the exposed reinforcing steel with respect to a crack is important. Where a crack is transverse to the reinforcement, only localized corrosion may occur. Research suggests that corrosion is limited to about 3 bar-diameters away from an intersecting crack, but during recent laboratory studies, the authors found significant corrosion as far as 13 bar-diameters [130 mm (5 in.)] away from the crack location. Concrete subsidence from vibration and consolidation during construction is common below the reinforcing steel, and this void below the bar provides a path for chloride-containing water, and corrosion may occur along the bar length even though the crack is transverse to the bar. However, deck cracking commonly appears directly over the reinforcing, because the bars delineate a weakened plane. This increases the potential for corrosion of reinforcing steel along the length of the bars, because the crack exposes a large area of the bar. Where the crack coincides with the bar, the passivity is lost at many locations and corrosion can proceed at an increased rate.

Cracks may reduce the durability of epoxy-coated reinforcement. Recent laboratory studies (*1*) suggest that concrete cracking may deteriorate epoxy coatings. These studies indicate that epoxy coating greatly enhances the corrosion resistance of reinforcing steel, but corrosion can occur when coating defects are present. If the deck cracks intersect the epoxy-coated bar in an area containing a break or holiday in the coating, corrosion can occur.

Concrete with cracks that allow water penetration can saturate quicker than uncracked concrete. This will reduce the durability of marginally air-entrained concrete during winter freezing and thawing conditions. Concrete with reactive or unstable aggregates will also be less durable because of cyclic wetting and drying.

Cracked decks subjected to deicing or aggressive solutions should be repaired to prevent infiltration through the cracks. Epoxy and high molecular weight methacrylate (HMWM) resins are used to bond the cracks structurally, by either pressure injection or topical gravity feed. Epoxy injection is commonly used for large cracks, and HMWM is commonly used for fine cracks on bridge decks. If a crack is not structurally significant, but it moves during cyclic temperature changes or other causes, it may be treated as a joint, and routed and sealed with an elastomeric sealant. For most new bridges, it would be very cost-effective to repair all visible deck cracks subjected to deicing chemicals.

## RESEARCH APPROACH

The research part of this study included a literature search, a survey of transportation agencies, instrumentation of a bridge deck replacement, analytical studies, and laboratory testing.

### Literature Review

The extensive literature review examined relevant domestic and foreign literature, research findings, performance data, and current practices concerning the causes and prevention of transverse cracking in bridge decks. This review used major reference databases, the National Technical Information Service (NTIS), Dialogue, American Concrete Institute (ACI) abstracts, and technical libraries. The list of the most relevant articles and papers reviewed are referenced and discussed in Appendix A, which is not published herein.

The reviewed literature examines the characteristics of transverse cracking that may accelerate corrosion of reinforcing steel, deteriorate concrete, damage structural members and components beneath the deck, and detrimentally affect visual appearance. This project studied the opinions and research of many authors and committees regarding harmful crack widths.

### Transportation Agency Survey

An extensive survey of transportation agencies determined the extent of the concrete deck cracking. A survey was sent to all U.S. DOTs and several transportation agencies overseas to collect performance data and published or unpublished reports relating to experience with deck cracking. Many of these agencies had already conducted surveys on the extent of longitudinal and transverse cracking in bridge decks. The results of a North Carolina DOT survey in 1979 and an AASHTO survey in 1990 are discussed in Appendix B, which is not published herein. It is apparent from the reviewed literature and the recent survey that cracking in new bridge decks is a significant national and international problem.

### Laboratory Testing

Because concrete mix design affects cracking, a test procedure was developed to compare the cracking tendency of different concretes. A description of the test procedure is provided in this report. The test results are presented in Appendixes C and D, which are not published herein. In brief, the test measures the time required for a concrete annulus to crack when an inner steel ring restrains it, similar to the way steel or concrete girders restrain a deck.

The procedure determines the effects of mix variations on the time required for cracking to develop. Mix variations that affect cracking include aggregate type and gradation, cement type and amount, water content, mineral admixtures, silica fume admixtures, and chemical admixtures. The procedure cannot determine when cracking will start in a specific type of structure, because actual cracking in a structure depends on many variables including restraint, hydration effects, and environment. However, the method can determine the relative likelihood of early concrete cracking, and aid in the selection of concrete mixes less likely to crack. The test can also evaluate the effect of environmental and construction factors by modifying the test environment or curing procedures.

### Bridge Deck Instrumentation

One bridge was instrumented during construction to study, from casting to post-cracking, the shrinkage and thermal behavior of a new bridge deck. The Portland-Columbia Bridge, located between Pennsylvania and New Jersey on Route 512, was instrumented and tested during its deck replacement in 1992. Details of the instrumentation and monitoring system are presented in Appendix E, which is not published herein. The information obtained from the bridge deck instrumentation was compared with theoretical (analytical) behavior.

### Analytical Studies

The analytical studies used conventional and finite element analysis techniques to evaluate the influence of various factors on deck tensile stresses and cracking. These factors included concrete drying shrinkage, creep, hydration temperatures and other thermal effects, position and amount of reinforcing steel, girder size and spacing, single- and multispan conditions, age, and other parameters. The individual and combined effects of material properties and geometries on stresses in concrete bridge decks were analyzed. This parameter study analyzed bridges with steel, reinforced concrete, precast, prestressed concrete, and cast-in-place, post-tensioned girders. Simple-span and continuous two-span structures were studied. Three different temperature conditions (either an increase or a decrease) and two different deck drying shrinkage profile conditions were studied. For steel-girder bridges, 2270 combinations of geometry and materials properties were analyzed for each of the three different temperature change conditions; while 2264 combinations were analyzed for each of the three temperature changes for the three types of concrete bridge systems. For drying shrinkage, 1134 combinations were analyzed for the two different deck shrinkage profiles for each of the steel- and concrete-girder system bridges. About 18,000 bridge system scenarios were analyzed. These analytical techniques enabled a comprehensive investigation of the parameters that affect cracking. Results and details of the analytical studies are presented in Appendixes F, G, and H, which are not published herein.

### General Remarks

Much of this project focused on the identification, evaluation, and ranking of factors or combinations of factors that contribute to transverse cracking of new bridge decks. The factors are categorized by design, materials, and construction. From an understanding of the mechanisms of bridge deck cracking, methods for crack prevention are proposed.

As part of this project, Guidelines were prepared that include recommendations to prevent or reduce transverse cracking in new bridge decks. These Guidelines, published herein, are designed for the specific needs of bridge designers, contractors, and material engineers. The Guidelines contain recommended practices for design, construction, and material selection, to reduce the occurrence of transverse deck cracking.

## APPLICABILITY OF RESULTS TO HIGHWAY PRACTICE

This project was structured to benefit bridge design and construction immediately. This report and the Guidelines provide a state-of-the-art review of the factors that contribute to deck cracking. The Guidelines can be used to modify current specifications to reduce cracking.

AASHTO should consider adoption of the proposed cracking-tendency test. It can be incorporated into contract specifications or used to identify concrete mixtures and construction procedures that reduce cracking. Because structural design of bridges, concrete specifications, and available materials continuously change, this test procedure will be particularly useful to transportation agencies.

CHAPTER 2

# RESEARCH FINDINGS

## FINDINGS OF THE SURVEY OF TRANSPORTATION AGENCIES

A survey was sent to U.S. DOTs and other transportation agencies to learn current design and construction practice, and the perceived causes of transverse deck cracks. The information gained from the survey guided the analytical and concrete studies for this project.

### Findings of Survey of North American Transportation Agencies

Fifty-two agencies in the United States and Canada replied to the survey. Sixty-two percent of the agencies considered early transverse cracking to be a problem, 24 percent did not consider it a problem, and 14 percent had no opinion. How DOTs view transverse cracking is shown in Figure 3. Many states that do not consider transverse cracking to be a problem nonetheless reported extensive cracking.

The 52 agencies reported having more than 225,000 bridges. The percentage of decks perceived by these agencies to be cracked varied widely and averaged 52 percent, with a standard deviation of 33 percent. Fifteen percent of the agencies believed 100 percent of their decks suffered from transverse deck cracking.

The age at which cracking occurs also varied widely. The respondents reported, on average, that 42 percent of decks cracked within the first week (standard deviation 32 percent), 53 percent cracked within the first month (standard deviation 34 percent), and 68 percent cracked within the first year (standard deviation 31 percent). Seventy-one percent visually inspect decks for cracking shortly after construction, and 4 percent pond the decks with water to check for cracking.

Thirty-six percent reported cracking primarily over the top transverse reinforcing bars. Thirteen percent reported cracking both over and between the transverse top bars, and only one agency (2 percent) reported cracking only between the top bars. The remaining agencies (49 percent) did not know if cracks align with the steel reinforcement.

Thirty-three percent of the respondents related cracking to the time of day the concrete was placed. Eighty percent of these believe that cracking is worse with afternoon placements. Sixty percent believe that less cracking occurs when concrete was cast during the evening. None said that cracking was worse during evening castings.

Only two agencies limit crack widths in new decks. One specifies that crack widths should be 0.18 mm (0.007 in.) or narrower, and the other specifies that not more than 15 m (50 ft) of cracks should be wider than 0.5 mm (0.020 in.) per 46.5 $m^2$ (500 $ft^2$) of deck area. Cracks typically are repaired using a penetrating sealer, HMWM, epoxy injection, or routing and sealing.

Most agencies (92 percent) follow AASHTO methods for design, and 57 percent follow AASHTO construction specifications. Typical structural design geometries were also surveyed. The survey determined the following regarding the use of different girder types:

- 88 percent design steel girders;
- 75 percent design precast, prestressed concrete girders;
- 33 percent design cast-in-place, post-tensioned girders; and
- 27 percent design conventionally reinforced, cast-in-place concrete girders.

Some North American transportation agencies perceived deck cracking to be worse with steel girders than with concrete girders, and some agencies perceived cracking to be worse in continuous structures than in simply supported structures.

When steel girders support the deck, studs are commonly attached to the upper flange of the girder. These studs typically are 19 or 22 mm (¾ or ⅞ in.) in diameter, 100 to 150 mm (4 to 6 in.) in length, and spaced longitudinally at 300 to 600 mm (12 to 24 in.). Three or four studs are typically used per row.

Of the transportation agencies responding, 80 percent typically require epoxy-coated bars, and 73 percent require epoxy-coated tie wires with epoxy-coated bars.

Two-layer deck construction is rarely used; however, many of the transportation agencies that use this construction reported problems with cracking.

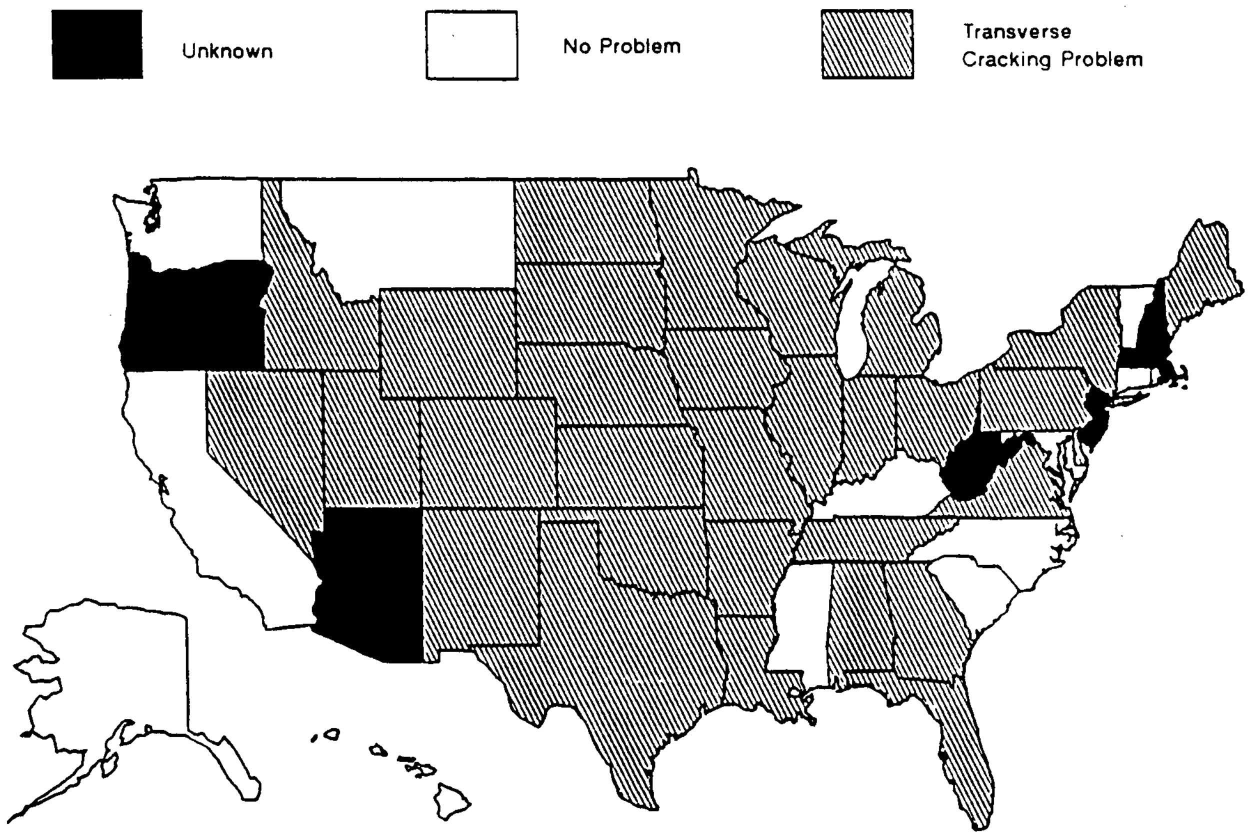


*Figure 3. How DOTs view transverse cracking.*

Seventy-seven percent construct decks with removable wood forms, 46 percent use stay-in-place (SIP) steel forms, and 20 percent use SIP precast concrete forms. A few agencies have stopped using SIP steel forms, but they did not give their reasons.

Seventy-seven percent specify a pouring sequence for continuous structures. Several agencies stated that pouring sequences are incorporated in the structural design.

Maximum permitted air temperatures during concrete placement vary significantly. Forty percent do not specify a maximum air temperature, 13 percent specify 27°C (80°F), 11 percent specify 29.5°C (85°F), 20 percent specify 32°C (90°F), and 5 percent specify 35°C (95°F). Minimum permitted air temperature during placement also varies significantly. Only one responding agency (2 percent) does not specify a minimum air temperature, 17 percent specify 1.5°C (35°F), 26 percent specify 4.5°C (40°F), 2 percent specify 7°C (45°F), 11 percent specify 10°C (50°F), and 42 percent did not respond.

Twenty-nine percent of the agencies specify a maximum concrete temperature at placement of 27°C (80°F), and 46 percent a maximum concrete temperature of 32°C (90°F). Eight percent specify a minimum concrete placement temperature of 7°C (45°F), 53 percent specify 10°C (50°F), and 13 percent specify 15.5°C (60°F). Only 6 percent of the agencies do not allow heating or cooling of aggregates and concrete.

Only one agency limits the relative humidity of the air, to a minimum of 35 percent. Twenty percent stipulate limits on the evaporation rate. Most specify an evaporation limit of 1 kg/m$^2$/hr (0.2 lb/ft$^2$/hr), except one agency specifies an evaporation limit of 0.75 kg/m$^2$/hr (0.15 lb/ft$^2$/hr); however, another agency specifies 0.5 kg/m$^2$/hr (0.10 lb/ft$^2$/hr). Seventy-seven percent believe that wind increases cracking, while 11 percent do not.

Seventy-one percent stated that contractors always strike-off the concrete perpendicular to the axis of the bridge, and 13 percent always strike-off parallel to the bridge axis. Eleven percent limit external vibration from sources such as traffic during curing.

Curing procedures vary greatly. At the time of the survey, several agencies specified linseed-oil curing, but its use is decreasing. Also, the time when curing commenced after strike-off varied significantly, and many agencies do not use consistent moist curing procedures. Types of curing, percentage of transportation agencies allowing the use of each type of curing, and the time of application are summarized below.

| Type of curing | Agency use | Time of application |
|---|---|---|
| Monomolecular film | 9 percent | Immediately or as soon as possible |
| Clear curing compound | 17 percent | Immediately or as soon as possible<br>As soon as the bleed water disappears<br>Half-hour after strike-off<br>After tining<br>Time not specified |
| Pigmented curing compound | 53 percent | Same as for clear curing compound |
| Fogging | 46 percent | Immediately or as soon as possible<br>Before strike-off<br>During strike-off<br>Half-hour after strike-off<br>After initial set<br>Four hours after strike-off |
| Wet burlap or fabric | 100 percent | Immediately after strike-off<br>As soon as possible<br>As soon as materials can be placed without damage to the concrete<br>As soon as the surface can be walked on<br>After initial set<br>Zero to half-hour<br>Half-hour<br>One hour<br>Four hours (provided curing compound is used immediately)<br>The following day<br>By noon the next day<br>Not specified |
| Plastic sheeting | 48 percent | Immediately<br>As soon as possible<br>As soon as finishing is completed<br>As soon as materials can be placed without damage<br>Half-hour<br>One hour<br>After initial set<br>Four hours<br>Next morning<br>Not specified |

The total curing time varied greatly, as shown below.

| Cure period | Number of agencies |
|---|---|
| 3 days | 4 percent |
| 5 days | 24 percent |
| 7 days | 53 percent |
| 10 days | 4 percent |
| 14 days | 2 percent |
| Not reported | 13 percent |

Transportation agencies specify various concrete properties. Thirteen percent specify a minimum compressive strength for bridge deck concrete of 24.1 MPa (3500 psi), 48 percent specify 27.5 MPa (4000 psi), 31 percent specify 31.0 MPa (4500 psi), and 8 percent did not report a minimum strength. Forty-five percent of the agencies specify a maximum water-cement ratio of 0.44 percent, and 15 percent specify a maximum water-cement ratio of 0.46. Forty-four percent and 36 percent of the agencies specify a maximum concrete slump with-out superplasticizers of 75 mm (3 in.) and 100 mm (4 in.), respectively. A typical paste to aggregate volume of 0.3 to 0.4 is used. Types I and II cement are commonly used. Three agencies sometimes specify Type III cement and one agency specifies Type K expansive cement.

Forty-nine percent specify crushed aggregate and 20 percent specify river gravel. There was significant variation in the maximum aggregate size; 28 percent specify 20 mm (0.75 in.), 31 percent specify 25 mm (1 in.), and 20 percent specify 37.5 mm (1.5 in.). Thirty-one percent specify that ready-mix supplier should maintain aggregate in a wet condition, and 55 percent do not specify wetted aggregates. Fifty-five percent use continuous-graded aggregates, and 31 percent use gap-graded aggregate.

Most agencies allow water-reducing admixtures and retarding admixtures. The percentages of agencies that allow the use of each admixture are shown below.

| Admixture | Percent permitting |
|---|---|
| water-reducing admixtures | 93 |
| retarding admixtures | 88 |
| superplasticizers | 60 |
| accelerators | 42 |
| calcium nitrite | 26 |

Of those that use superplasticizers, 58 percent allow job-site re-dosing. The maximum slump permitted by the agencies using superplasticized concrete varies significantly, as shown below.

| Maximum slump | Percent permitting |
|---|---|
| 125 mm (5 in.) | 20 |
| 150 mm (6 in.) | 30 |
| 175 mm (7 in.) | 20 |
| 200 mm (8 in.) | 30 |

Seventy-one percent allow fly ash admixtures, 28 percent allow silica fume admixtures, and 17 percent allow blast furnace slag admixtures.

Transportation agencies measure compressive strengths at various ages. Of those responding, 13 percent measure strengths at 3 days, 44 percent at 7 days, 17 percent at 14 days, and 75 percent at 28 days. Twenty-two percent of the agencies typically conduct flexural strength tests to determine the modulus of rupture (MOR), and almost all agencies test the slump and air content of delivered concrete. Fifty-seven percent measure the concrete unit weight, and 22 percent measure the concrete absorption.

Transportation agencies do not typically measure the modulus of elasticity, shrinkage, creep, cracking tendency, and permeability of concrete. Eight percent measure setting times, 6 percent measure heat of hydration, and 4 percent test aggregate for shrinkage. Many agencies do petrographic and chemical tests on aggregates to assess their suitability for use in concrete.

The survey also requested comments regarding the transportation agencies' perception of the causes of cracking in bridge decks. Survey results are summarized below. Several causes necessarily overlap; for example, curing practices will be influenced by environmental conditions such as wind, temperature, and humidity. The number in parentheses is the number of responses received that consider the factor to be a cause of cracking.

| Construction | Materials | Design |
|---|---|---|
| improper curing (20) | concrete shrinkage (17) | deflections (7) |
| wind (7) | [5 cited drying shrinkage specifically] | excessive cover (3) |
| thermal effects (7) | concrete mix design (7) | placement sequence (2) |
| air temperature (7) | plastic shrinkage (3) | |
| relative humidity (4) | excessive cement (3) | |
| vibration (2) | concrete temperature (3) | |
| placement conditions/weather (2) | use of retarders (2) | |

Other factors mentioned included poor construction, overfinishing, high-slump concrete, high water-cement ratio concrete, superstructure type, early form removal, fly ash additions, and camber or prestress changes.

Overall, there was a significant concern about curing practices. This is not surprising because this survey shows that these transportation agencies permit a wide range of curing procedures. Material factors such as drying shrinkage, plastic shrinkage, amount of cement, and use of retarders are also of concern to many of these agencies. Environmental conditions such as thermal effects and relative humidity are a concern as well. Construction practices (apart from curing and environmental conditions) and design practices (apart from deflections) are not considered major factors.

## Findings of Survey of Overseas Transportation Agencies

This project surveyed several overseas agencies and the results are summarized in this section.

**Australia**—The Roads and Traffic Authority in New South Wales observed early transverse deck cracking in the early 1980s. The Authority believed that insufficient temperature steel in the decks caused this cracking. Since then, the Authority has increased the amount of top steel reinforcement of the decks to 4000 $mm^2/m^2$ (0.4 percent) of concrete cross section. This reportedly has significantly reduced cracking. Reinforcing bars typically are 16 to 20 mm (0.63 in. to 0.75 in.) in diameter, and wet curing is specified. The effect of deck pour sequence is also considered during the design phase. Bridge design is based on Australian Standard AS3600, Concrete Structures (*2*), and the National Australian Association of State Road Authorities Bridge Design Code (*3*).

**Denmark**—The Danish Road Directorate, responsible for approximately 2000 bridge decks, considers transverse cracking to be a minor concern. Most of its bridges are cast-in-place, reinforced or post-tensioned concrete-girder designs. The Directorate has observed fine cracks in a few decks, attributed to temperature gradients and shrinkage. It has occasionally observed similar cracks in edge beams cast after the deck. Deck cracking has not required remedial attention. Top and bottom steel covers are typically 30 mm (1.2 in.), and a maximum bar size of 25 mm (1 in.) is used. The top and bottom bars are secured to each other, but are not aligned. Epoxy-coated bars are not used.

Acceptable crack widths are limited to 0.1 mm (0.004 in.) for prestressed concrete, and 0.2 mm (0.008 in.) for conventionally reinforced concrete. A visual inspection is conducted after construction. During casting, the Directorate specifies a minimum air temperature of −5°C (23°F) but not humidity or evaporation limits.

The Directorate specifies a minimum concrete strength of 35 MPa (5100 psi), a maximum water-cement ratio of 0.45, and a maximum aggregate size of 32 mm (1.25 in.). It allows silica fume, fly ash, air-entraining admixtures, and superplasticizers. Contractors may cure concrete with clear curing compounds, wet burlap, or plastic sheeting. The period of moist curing varies.

**Japan**—The authors received many surveys and letters from agencies and universities in Japan. Prof. Kokubu, from the University of Tokyo, stated that transverse cracking occurs in concrete slabs in Japan. Preventive measures used include thorough curing, the use of fly ash and other admixtures, and the use of steel fibers.

Mr. Sakai, of the Civil Engineering Research Institute, Hokkaido Development Bureau, also considers transverse cracking to be a problem. The design codes of the Japanese Road Institute, the Hokkaido Development Bureau, and the Japanese Society of Civil Engineers are used. The typical acceptable crack width is 0.2 mm (0.008 in.) or less. He has observed cracking both over and between reinforcing bars. After construction, decks are visually inspected for cracking. Cracking is typically worse on continuous-span bridges.

Mr. Sakai suggests that if the casting sequence is not properly considered, cracking problems will occur. Recommended air temperature at the time of placement is from 5°C (40°F) to 30°C (85°F). Humidity and evaporation limits for casting are not specified. He considers windy days more conducive to cracking. Japan requires that bridge decks have a concrete strength of 24 MPa (3500 psi), and a maximum water-cement ratio of 0.55. The typical slump specified is 80 mm (3.25 in.), and the maximum aggregate size is 20 to 25 mm (0.75 to 1 in.). Silica fume; fly ash; blast furnace slag; and air-entraining, water-reducing, retarding, and superplasticizing admixtures are used. Curing periods of more than 3 days are used. Monomolecular film, curing compounds, fogging, and burlap are used for evaporation control and subsequent moist curing. Mr. Sakai suggests that besides casting sequence, other major influences on deck cracking are curing and settlement of fresh concrete after placing.

**Switzerland**—The Federal Department of Highways in Switzerland considers transverse cracking of concrete bridges to be a problem among its 3000 bridges. The causes identified are restrained plastic and drying shrinkage. The Department suggests that transverse cracking cannot be prevented with certainty; however, adequate longitudinal reinforcement leads to good crack distribution and limits crack width to 0.2 mm (0.008 in.) or less, which is considered acceptable. All bridge decks are treated with a waterproofing system to prevent the penetration of deicing salts.

## FINDINGS OF THE LITERATURE REVIEW

The authors reviewed many reports on transverse cracking of concrete bridge decks. A list of the reviewed literature is given in the references.

The literature review shows that both excellent- and poor-quality bridge decks can be constructed. Deck cracking is not confined to one geographic location. Cracking generally becomes more severe with time and is often not visible during construction. Efflorescence has been observed on structures not yet opened to traffic. Decks that are not visibly cracked when the forms are removed usually develop visible cracks after traffic is allowed on the structure. Transverse cracks are commonly observed above the reinforcing bars, are usually full-depth, and are spaced between 1 and 3 m (3 and 10 ft) apart.

Cracks as narrow as 0.05 mm (0.002 in.) can allow water and chloride ion penetration. Transverse cracking can lead to concrete spalling at intersections with non-transverse cracks. Early transverse cracking does not occur because of corrosion, but such cracking may accelerate corrosion. This deterioration is particularly severe when black reinforcing bars are used. Researchers have also observed minor corrosion damage in epoxy-coated reinforcing bars at crack locations (*4*).

The literature review indicates that the most significant concrete material factors affecting transverse cracking are cement content, creep, elastic modulus, concrete temperature during placement, heat generated during hydration, drying shrinkage, and water content. Aggregate type, mineral additions, admixtures, and cement type also influence cracking.

General recommendations from the literature concerning concrete material properties to reduce cracking include using the following:

- Low amounts of cement;
- Good-quality, low-shrinkage aggregates;
- Air entrainment;
- Low drying shrinkage concrete;
- Moderate placement temperatures;
- Means to reduce hydration temperature rise,
- Low water content (water-cement ratio between 0.41 and 0.49); and
- Type II cement.

Several transportation agencies suggested that shrinkage-compensating cement reduces deck cracking. However, results of laboratory and field investigations relating shrinkage-compensating cement and early cracking are mixed. There is controversy on the use of retarders, accelerators, fiber reinforcement, fly ash, and silica fume, and their roles in deck cracking; further research is required.

The literature indicates that the major design factors that affect cracking are restraint, structure type, concrete properties, and girder type. Other potential factors include dead-load deflection, concrete cover, span length, and reinforcement.

Cracking was found more prevalent on continuous spans than on simple spans (*5*). Some researchers found cracking worse on structures supported by steel girders than on those supported by concrete girders (*5,6*); however, cracking has been observed on both types of structures (*5,6*). One researcher (*7,8*) found cracking worse with precast girders than with cast-in-place girders, and worse for steel girders when SIP steel forms are used. Decks supported on wide-flange steel beams or composite steel plate girders cracked much more than those constructed on other systems. (*5*) One researcher (*9*) reported that structures with skews greater than 30 degrees are more susceptible to transverse cracking, particularly near corners.

General conclusions concerning design parameters include the following:

- Cracking is more common on steel girder structures.
- Continuous-span structures are more susceptible to cracking than simple-span structures.
- Longer span decks are more susceptible to cracking.
- Thermal stresses may be significant and cause early cracking.
- Dead-load deflections during construction should be considered during design.
- Cover over reinforcement should be between 25 mm (1 in.) and 76 mm (3 in.).
- Decks should not be less than 200 to 230 mm (8 to 9 in.) thick.
- Use of epoxy-coated bars increased the width of deck cracks.
- More bars of smaller diameter reduce crack widths.
- Increasing the amount of reinforcement may reduce the crack widths.

The literature indicates that weather during construction affects cracking. Adverse conditions include high winds, extreme low and high temperatures, and low humidity. Concrete should not be placed during periods of high evaporation, unless evaporation retarder films, sun shades, wind breaks, and fogging are used. Casting at night significantly reduces deck cracking (*6,9*), and afternoon placements are most likely to crack.

The transportation agencies most commonly cite inadequate curing as the major cause of deck cracking. Effective curing should commence as soon as possible. Evaporation film retarders or fogging applied even before finishing can significantly reduce the number of small deck cracks that form during hot or cold conditions. Wet curing should be considered during hot weather to cool the concrete and reduce peak concrete temperatures. The period of moist curing should be at least 14 days. A survey of pavements cured with clear membranes found that cracking occurred predominantly in pavements laid in morning hours (*10*).

Pour size and pour sequence were not considered significant in affecting transverse cracking (*9, 11*). Traffic-induced vibrations during hardening were also not found detrimental (*7,8,12,13*). Adequate initial vibration is necessary to prevent settlement cracking. Construction personnel should also be aware of cracks in plastic concrete during finishing. These may be revibrated and closed while the concrete is still plastic.

Each material, design, and construction factor is described in detail later in this chapter, including the findings of all portions of this study as they relate to each factor.

## FINDINGS OF THE FIELD STUDIES

An extensive instrumentation and monitoring system was designed and installed on the Portland-Columbia Bridge between Pennsylvania and New Jersey. This system measured strains and temperatures in the bridge deck and girders, starting when the new deck concrete was cast and continuing for several months. The system also monitored environmental conditions. The collected data in the field studies were analyzed in detail in the analytical studies.

Many concrete test cylinders were made from the concrete used in the new deck. Concrete compressive and splitting tensile strengths, modulus of elasticity, and unrestrained free-shrinkage properties were measured at various times. The deck concrete was a 0.41 water-to-cement ratio concrete that produced a nominal 28-day compressive strength of 31.0 MPa (4500 psi). A high range water reducing admixture (HRWRA) was used and the slump was 130 to 190 mm (5 to 7½ in.). By combining measurements of deck strain, temperature, environment, concrete properties, and incidence of cracking, valuable information was obtained about the changing state of strain and other conditions.

The data collected do not necessarily reflect conditions at other bridge decks. However, the data collected verified that theoretical analysis (1) can predict actual behavior and (2) provides a better understanding of early bridge behavior.

The Portland-Columbia Bridge opened in 1953. It carries two lanes of traffic across the Delaware River between Portland, Pennsylvania, and Columbia, New Jersey. It is owned and operated by the Delaware River Joint Toll Bridge Commission (DRJTBC), Morrisville, Pennsylvania.

The bridge consists of eight spans of 45 m (147 ft 6 in.), two end spans of 19.3 m (63 ft 4 in.), and a west abutment span of 12.8 m (42 ft 2 in.). The girders in all spans are simply supported. The superstructure consists of four built-up riveted steel girders spaced 2.6 m (8 ft 8 in.) apart, and a 200-mm (8-in.) thick, reinforced concrete deck. Total width of the original bridge deck was 9.7 m (32 ft), consisting of a 7.6-m (25-ft) wide roadway, a 1.5-m (5-ft) wide south sidewalk and a 0.6-m (2-ft) wide north sidewalk. Because of an ever increasing incidence of bridge deck delamination, the DRJTBC decided in 1991 to replace the original bridge deck and make other improvements.

The new deck is also 9.7 m (32 ft) wide, consisting of an 8.8-m (29-ft) wide roadway and 460-mm (1-ft 6-in.) wide "New Jersey" barriers along the north and south edges. The new deck is 200- to 250-mm (8- to 10-in.) thick. It is reinforced with two layers (top and bottom) of 20-mm (No. 6) bars placed 180 mm (7 in.) on-center in the transverse direction, a bottom layer of 16-mm (No. 5) bars at 220-mm (8½ in.) spacing in the longitudinal direction, and a top layer of 16-mm (No. 5) bars at 290-mm (11½-in.) spacing in the longitudinal direction. The reinforcing bars are epoxy-coated. Between the girders the new deck is cast on SIP steel deck forms; elsewhere, it is cast on plywood forms.

The Portland-Columbia Bridge is ordinary in most respects, with the possible exception of its light traffic volume. This bridge is representative of many steel girder bridges, especially those in similar geographical regions.

In addition to measuring the strain of restrained concrete ring and free-shrinkage prism specimens, instrumentation was installed to measure the following:

- Strain and temperature in the concrete deck,
- Strain and temperature of the steel girders,
- Rotation of a steel girder at its bearings,
- Wind speed and direction,
- Air temperature and relative humidity,
- Solar radiation and water evaporation rate, and
- Vibration.

To study the thermal behavior of the Portland-Columbia Bridge, data were recorded during three time periods. The first period consisted of the first 4 days after casting, during which significant heat of hydration developed; the largest temperature differences and gradients occurred during this period. The second period consisted of the first 21 days, when wet curing stopped at the eighth day and the concrete matured. The third period consisted of 7 days when the concrete was approximately 1 month old, during which it did not rain and temperatures cycled similarly each day. The shrinkage-behavior study examined all data but did not examine a specific period. Traffic was not allowed onto the bridge until June 26, 1992 (day 43).

The largest temperature changes in the Portland-Columbia Bridge occurred within 48 hrs from the time the concrete deck was placed. During the first 12 hrs, temperatures in the new deck climbed from 27°C (80°F) to as high as 55°C (131°F) from the exothermic reaction of the cement. Temperatures in the deck varied substantially along the length and across the width of the bridge.

After 48 hrs, the temperature differentials had substantially dissipated, along with the measured strain differentials in the restraining steel members. Creep of the young concrete had dissipated most, but not all, of the strain caused by hydration temperatures. Concrete compressive strengths at 6, 11, and 27 hrs were about 1.2 MPa (180 psi), 10.8 MPa (1600 psi) and 16.1 MPa (2300 psi), respectively.

A similar temperature decrease then followed this hydration temperature increase, as the concrete cooled to temperatures equal to ambient air temperature. This temperature drop caused tensile stresses in the hardened concrete, as the steel girders, reinforcing bars, and SIP metal deck restrained the thermal contraction. Concrete strength at 48 hrs was about 18.0 MPa (2600 psi) and the concrete modulus of elasticity was about 19.3 GPa ($2.8 \times 10^6$ psi). This modulus value at 48 hrs was already 82 percent of the 28-day modulus of elasticity, an important factor in development of restrained tensile stresses resulting from thermal contractions and subsequent drying shrinkage.

About 2 days after placement, temperatures in the bridge were nearly uniform, but strain and stresses were not; cracking did not occur because thermal stresses did not yet exceed the tensile strength of the concrete. About 4 days after placement, strains in the deck became nearly uniform as creep of the young concrete dissipated most of the early nonuniform thermal effects. It is unlikely that early thermal stresses alone caused cracking of the Portland-Columbia Bridge, because later cracking was nearly uniform along the length of the bridge and was not limited to specific areas that had extreme temperatures. If a bridge deck does not crack within the first few days after placement, creep may reduce thermal stresses caused by hydration. However, these early stresses may cause micro-cracking and weakening of the young concrete, contributing to later cracking from other temperature changes and shrinkage.

Visual examinations of the upper surface of this moist-cured concrete deck revealed that significant cracking of this surface first occurred between June 9 and July 2, 1992, (days 26 and 49) after 18 to 41 days of air drying. However, strains recorded by the strain gages on the embedded reinforcement show that cracking probably occurred below the top surface—by May 26 (day 12), after only 4 days of air drying—but had not yet propagated to the upper surface. The strain gages on the reinforcement provided the best indication of when cracking occurred, because the Carlson gages embedded in the concrete did not record the same sudden strain changes. The strain gages on the reinforcement suggested that additional cracking over the interior girder continued to occur from July 9 (day 56) through July 20 (day 67), moving from midspan toward the ends. When cracking was suspected at 12 days, the concrete modulus of elasticity was about 21.4 GPa ($3.1 \times 10^6$ psi) and represented 90 percent of the 28-day modulus of 23.6 GPa ($3.4 \times 10^6$ psi).

The instrumented span and two adjacent spans were visually inspected on day 98 for crack widths and number of cracks. Sixteen to 17 major transverse cracks were found on the top surface of each of these three spans. The SIP steel forms prevented bottom surface inspection. The crack widths of 11 of the 16 major transverse cracks on the instrumented span were measured. The average top surface crack width was 0.178 mm (0.007 in.), with a standard deviation of 0.046 mm (0.0018 in.). The average spacing between major cracks was 1.7 m (5.6 ft), with a standard deviation of 0.52 m (1.7 ft). The nominal 12-m (40-ft) crack spacing in the central region between cracks was not used in determining the average crack spacing. The central 12-m (40-ft) region of the instrumented span did not contain major transverse cracks but did contain small hairline transverse cracks that were relatively short and did not cross the deck.

The restrained concrete ring specimens, cast when the deck of the Portland-Columbia Bridge was placed, cracked when they were 36 days old. This time coincides with when the upper surface of the bridge deck became visibly cracked. This correlation suggests that cracking of the Portland-Columbia Bridge was caused primarily by shrinkage, because the ring specimens showed that thermal stresses were negligible and shrinkage caused cracking. Also, the early thermal strains in the deck and reinforcement dissipated because of creep. The correlation between the bridge deck and rings suggests that the ring test can predict when a bridge deck will crack. This correlation between deck cracking at 26 to 49 days and ring cracking at 36 days is supported by the results of the analytical studies and the concrete ring studies. The restrained concrete ring studies show that for a reasonable linear-shrinkage strain distribution through the concrete ring, the degree of restraint is about 70 percent. The analytical studies of actual bridges show that the following degrees of restraint occur for a reasonable linear-shrinkage strain distribution through the bridge deck:

- Steel girders typically restrain between 85 and 95 percent of the curvature effects.
- Concrete girders typically restrain 75 to 95 percent of the curvature effects.
- Larger girders restrain more curvature than small girders.
- Because of interface shear, the final degree of restraint in the deck is less than the curvature restraint suggests.
- Large steel or concrete girders can restrain about 60 percent of the free-strain at the upper surface of the deck, and smaller girders often restrain 35 to 45 percent of the free-strain at the upper surface.
- When linear free-strains are applied to most decks, bending stresses are larger than membrane stresses, and stress reversals develop in the deck.

Because the Portland-Columbia Bridge was built with large steel girders, the actual degree of restraint was probably about 60 percent, similar to the 70 percent offered by the steel ring. Therefore, similar cracking behavior could be anticipated.

The field data were used to develop and test the analytical models, as discussed in detail within the analytical studies.

## FINDINGS OF THE ANALYTICAL STUDIES

### Background

Longitudinal tensile stresses in a concrete deck cause transverse cracking. These stresses are largely caused by concrete shrinkage and changing bridge temperatures, and to a lesser extent by traffic. Bridge designers rarely examine nonuniform shrinkage and temperature effects in design.

Systems of equations were derived to calculate stresses in a composite reinforced concrete bridge subjected to uniform and linear temperature and shrinkage conditions. The equations consider multiple layers of reinforcement in the deck to account for the restraint effects of longitudinal deck reinforcement and a stay-in-place metal deck. Theoretical behavior predicted with these equations compared favorably with measured behavior of the Portland-Columbia Bridge. Although measured strains and temperatures across the width and length of the bridge were not uniform, average measured strains are similar to theoretical strains when appropriate concrete properties and shrinkage rates are applied. Therefore, the elastic equations can estimate average shrinkage and thermal stresses in a bridge, but actual stresses across the width and along the length will vary.

Using these systems of equations, designers can evaluate different designs and can select designs that reduce deck stresses and cracking. Because deck stresses and cracking are affected by the combination of geometry and material properties, this project examined shrinkage or thermal stresses in more than 18,000 bridge combinations of geometry and materials. This parameter study examined the stresses caused by uniform and nonuniform shrinkage and temperature changes in bridges, and determined how bridge geometry and material properties affect these stresses and the risk of transverse cracking.

Elastic stresses corresponding to a given strain are linearly proportional to the modulus of elasticity of the material. The modulus of elasticity of concrete is usually measured by using ASTM C469 procedures, before creep affects strains. This modulus of elasticity is appropriate to analyze short-term loading, but is inappropriate to analyze long-term strains such as shrinkage due to creep. When analyzing sustained strains, a lower modulus of elasticity is appropriate to calculate stresses and account for creep. This modulus of elasticity is called the "effective modulus of elasticity." The analytical studies examined effective concrete moduli between 3.4 and 31 GPa (0.5 and 4.5 $\times$ $10^6$ psi). The lowest value can characterize fresh concrete before it has crept, or it can model mature low-strength concrete with moderate or high creep. The larger value can represent mature concrete with little or no creep. An intermediate modulus of elasticity can represent mature concrete with low or moderate creep, or a high-strength concrete with moderate creep.

Shrinkage and temperature change can cause large tensile stresses in bridge decks. This project's parameter study found that decks of simply supported bridges with steel girders could develop stresses as large as the following:

| Applied temperature or shrinkage, simply supported steel-girder bridge | Maximum Deck Stresses | |
|---|---|---|
| | Tension | Compression |
| 28°C (50°F) uniform **increase**, full section | 1.9 MPa (284 psi) | 1.5 MPa (221 psi) |
| 28°C (50°F) uniform **decrease**, full section | 1.5 MPa (221 psi) | 1.9 MPa (284 psi) |
| 28°C (50°F) uniform **increase**, deck section only | 2.2 MPa (313 psi) | 5.9 MPa (861 psi) |
| 28°C (50°F) uniform **decrease**, deck section only | 5.9 MPa (861 psi) | 2.2 MPa (313 psi) |
| 28°C (50°F) linear **increase**, deck section only | 5.9 MPa (851 psi) | 9.3 MPa (1352 psi) |
| 28°C (50°F) linear **decrease**, deck section only | 9.3 MPa (1352 psi) | 5.9 MPa (851 psi) |
| 100 $\mu\epsilon$ uniform shrinkage | 1.9 MPa (279 psi) | 0.6 MPa (86 psi) |
| 100 $\mu\epsilon$ linear shrinkage | 2.8 MPa (407 psi) | 1.6 MPa (238 psi) |
| 500 $\mu\epsilon$ uniform shrinkage | 9.6 MPa (1395 psi) | 3.0 MPa (430 psi) |
| 500 $\mu\epsilon$ linear shrinkage | 14.0 MPa (2035 psi) | 8.2 MPa (1190 psi) |

Similarly, shrinkage and thermal stresses in the deck of a simply supported bridge with concrete girders can be as large as the following:

| Applied temperature or shrinkage, simply supported concrete-girder bridge | Maximum Deck Stress | |
|---|---|---|
| | Tension | Compression |
| 28°C (50°F) uniform **increase**, full section | 0.69 MPa (100 psi) | 1.38 MPa (200 psi) |
| 28°C (50°F) uniform **decrease**, full section | 1.38 MPa (200 psi) | 0.69 MPa (100 psi) |
| 28°C (50°F) uniform **increase**, deck section only | 2.59 MPa (375 psi) | 7.89 MPa (1144 psi) |
| 28°C (50°F) uniform **decrease**, deck section only | 7.89 MPa (1144 psi) | 2.59 MPa (375 psi) |
| 28°C (50°F) linear **increase**, deck section only | 5.70 MPa (827 psi) | 10.2 MPa (1480 psi) |
| 28°C (50°F) linear **decrease**, deck section only | 10.2 MPa (1480 psi) | 5.70 MPa (827 psi) |
| 100 $\mu\epsilon$ uniform deck shrinkage | 2.42 MPa (351 psi) | 0.72 MPa (104 psi) |
| 100 $\mu\epsilon$ linear deck shrinkage | 3.0 MPa (441 psi) | 1.5 MPa (219 psi) |
| 500 $\mu\epsilon$ uniform deck shrinkage | 12.1 MPa (1755 psi) | 3.59 MPa (520 psi) |
| 500 $\mu\epsilon$ linear deck shrinkage | 15.2 MPa (2205 psi) | 7.55 MPa (1095 psi) |

Bridges with concrete girders are affected more by continuity over bridge supports than bridges with steel girders. Because steel girders typically restrain more free-curvature of a deck than concrete girders, the bridge supports usually will restrain less curvature with steel girders. The following stresses can develop in the deck of a continuous-span bridge with steel girders:

| Applied temperature or shrinkage, multispan steel girder bridges | Maximum Total Stress | |
|---|---|---|
| | Tension | Compression |
| 28°C (50°F) uniform **increase**, full section | 2.0 MPa (291 psi) | 1.55 MPa (225 psi) |
| 28°C (50°F) uniform **decrease**, full section | 1.55 MPa (225 psi) | 2.0 MPa (291 psi) |
| 28°C (50°F) uniform **increase**, deck section only | 1.38 MPa (200 psi) | 6.08 MPa (882 psi) |
| 28°C (50°F) uniform **decrease**, deck section only | 6.08 MPa (882 psi) | 1.38 MPa (200 psi) |
| 28°C (50°F) linear **increase**, deck section only | 0.10 MPa (14 psi) | 9.73 MPa (1412 psi) |
| 28°C (50°F) linear **decrease**, deck section only | 9.73 MPa (1412 psi) | 0.10 MPa (14 psi) |
| 100 $\mu\epsilon$ uniform shrinkage | 1.96 MPa (284 psi) | 0.39 MPa (57 psi) |
| 100 $\mu\epsilon$ linear shrinkage | 2.91 MPA (422 psi) | 0.13 MPa (19 psi) |
| 500 $\mu\epsilon$ uniform shrinkage | 9.78 MPa (1420 psi) | 1.96 MPa (285 psi) |
| 500 $\mu\epsilon$ linear shrinkage | 14.54 MPa (2110 psi) | 0.59 MPa (95 psi) |

Similarly, decks of continuous-span bridges with concrete girders can develop the following stresses:

| Applied temperature or shrinkage, multispan concrete girder bridges | Maximum Total Stress | |
|---|---|---|
| | Tension | Compression |
| 28°C (50°F) uniform **increase**, full section | 0.27 MPa (39 psi) | 2.25 MPa (327 psi) |
| 28°C (50°F) uniform **decrease**, full section | 2.25 MPa (327 psi) | 0.27 MPa (39 psi) |
| 28°C (50°F) uniform **increase**, deck section only | none | 13.5 MPa (1958 psi) |
| 28°C (50°F) uniform **decrease**, deck section only | 13.5 MPa (1958 psi) | none |
| 28°C (50°F) linear **increase**, deck section only | none | 13.6 MPa (1969 psi) |
| 28°C (50°F) linear **decrease**, deck section only | 13.6 MPa (1969 psi) | none |
| 100 $\mu\epsilon$ uniform shrinkage | 3.86 MPa (560 psi) | none |
| 100 $\mu\epsilon$ linear shrinkage | 3.88 MPa (563 psi) | none |
| 500 $\mu\epsilon$ uniform shrinkage | 19.29 MPa (2800 psi) | none |
| 500 $\mu\epsilon$ linear shrinkage | 19.40 MPa (2815 psi) | none |

### Deck Shrinkage Stresses

Deck shrinkage stresses in simply supported spans are generally higher on steel-girder structures than those on concrete-girder structures. Deck shrinkage stresses are typically lowest in a monolithic cast-in-place bridge, where both the deck and girders shrink together. A bridge deck supported by precast, prestressed girders may develop more or fewer shrinkage stresses than the monolithic non-prestressed concrete bridge, depending on the shrinkage and creep of the precast girders and the age of the precast girders when the concrete deck is cast.

When the concrete deck shrinks relative to its girders in a simply supported span, 500 $\mu\epsilon$ uniform shrinkage of the concrete deck may cause tensile stresses as large as 9.65 MPa (1400 psi) in the deck of a steel girder bridge and 12.4 MPa (1800 psi) in the deck of a concrete bridge, depending on geometry and material properties; maximum stresses from a linear shrinkage profile, where 500 $\mu\epsilon$ occurs at the top and zero $\mu\epsilon$ at the bottom, are slightly larger. These stresses can cause transverse cracking.

Additional shrinkage stresses develop over the interior supports of continuous-span bridges. These additional tensile stresses are generally small in most bridges with steel girders. When concrete girders are used, these additional stresses are also generally small when the girders and deck shrink uniformly with depth. However, when differential shrinkage occurs between the concrete girders and the deck, total tensile stresses over the interior support may reach nearly 13.8 MPa (2000 psi) for differential shrinkage of 500 $\mu\epsilon$, sometimes substantially larger than the stresses away from the support.

### Hydration Temperature Stresses

Thermal stresses from early hydration temperatures are largest in a steel girder bridge, or in a concrete bridge when the deck is cast separately from the girders. A 28°C (50°F) temperature change in the deck relative to the girders can cause stresses greater than 1.38 MPa (200 psi) when the concrete has an early effective modulus of elasticity of only 3.5 GPa ($0.5 \times 10^6$ psi), and greater than 6.89 MPa (1000 psi) when the early effective modulus is 17.2 GPa ($2.5 \times 10^6$ psi).

### Diurnal Temperature Stresses

For most bridges, diurnal temperature changes produce larger thermal stresses than a seasonal temperature change produces. The parameter study analyses revealed that a linear (nonuniform) temperature change in the deck typically causes larger stresses than a uniform temperature change. For example, for most bridges, a linear temperature change in the deck of 28°C (50°F) from the upper surface to the soffit surface will produce larger stresses than a uniform 28°C (50°F) temperature change in the deck.

In a simply supported bridge, thermal tensile stresses from a 28°C (50°F) linear temperature change in the deck may reach 9.31 MPa (1350 psi) with steel girders, and 10.2 MPa (1480 psi) with concrete girders. Diurnal thermal stresses are often larger over the interior supports of a continuous span structure. Thermal tensile stresses above these interior supports may exceed 9.65 MPa (1400 psi) with steel girders and 13.8 MPa (2000 psi) with concrete girders. All these stresses are sufficient to cause transverse deck cracking, especially over interior supports of a continuous-span structure.

### Seasonal Temperature Stresses

Stresses from seasonal temperature changes are small or negligible in concrete bridges because both deck and girders typically have similar or the same thermal expansion coefficients. Deck stresses in concrete bridges caused by seasonal (uniform full-depth) temperature changes occur only because of expansion differences of the concrete and deck reinforcing steel.

When steel girders support the concrete deck, seasonal temperatures will cause thermal stresses when the concrete does not have the same thermal expansion rate as the steel. Because most concrete has a lower coefficient of thermal expansion than steel has, uniform seasonal temperature decreases will generally cause compressive stresses to develop in the deck, and uniform temperature increases will cause tensile stresses in the deck. A uniform full-depth temperature change in a steel bridge may cause stresses as large as 1.96 MPa (284 psi) in a simply supported span, and 2.01 MPa (291 psi) over interior supports of a continuous bridge.

### Factors Affecting Shrinkage and Thermal Stresses

Many factors affect shrinkage and thermal stresses. The primary factors include the concrete material itself, the geometry of the bridge, construction techniques, and the bridge environment.

Shrinkage stresses are generally linearly proportional to the shrinkage of the concrete. When shrinkage doubles, shrinkage stresses also may double if creep effects are similar. Concrete with a lower shrinkage will develop lower shrinkage stresses.

Thermal stresses from seasonal (full-depth) temperature changes are linearly proportional to the differences between material thermal expansion coefficients. For diurnal temperature changes affecting temperatures in the deck more than in the girders, thermal stresses are proportional to the concrete coefficient of thermal expansion. Because diurnal thermal stresses are generally larger than seasonal thermal stresses, combined thermal stresses usually increase when the concrete coefficient of thermal expansion increases.

Concrete aggregates affect shrinkage and thermal stresses, and therefore transverse deck cracking. Aggregates with a lower modulus of elasticity decrease the modulus of elasticity of the concrete, and the shrinkage and thermal stresses. These aggregates also often increase creep, further reducing shrinkage stresses. More thermally conductive aggregates may reduce thermal gradients within the deck, lowering thermal stresses.

Shrinkage stresses are affected less by geometry than by material properties. Generally, larger deck stresses develop with deep girders at a narrower spacing and with thinner decks. Deck reinforcement has a small effect on thermal stresses in the deck, typically increasing thermal and shrinkage stresses slightly. Steel studs or channels used with steel girders locally increase average deck stresses and may increase transverse deck cracking. A SIP steel form will (1) cause deck shrinkage that is more linear (nonuniform) than uniform, (2) produce larger tensile stresses at the upper surface of the deck, and (3) may increase the risk or severity of transverse deck cracking.

## FINDINGS OF THE CRACKING-TENDENCY TESTING

This project developed a cracking-tendency test to compare various concrete mixtures, curing, and environmental factors.

### Background

Others have used many different test configurations, ranging from restrained linear prisms to thick concrete rings, to investigate shrinkage and cracking of different concretes. The authors investigated the limitations of the different test methods to determine the most suitable test for future use. The threaded rod linear-type prism system originally used by Carlson (*14*) is limited because it requires a bond between the concrete and the rod. Because time is required to develop the bond, and because only about 50 percent restraint develops, it is difficult to determine the effects of plastic shrinkage and autogenous volume changes. Carlson (*14*) and other researchers (*15,16*) successfully used a ring test to indicate whether a particular concrete mixture is susceptible to cracking. This type of test best predicts cracking, and was used for this project to study the cracking tendency of various concretes.

Past research suggests that the type of coarse aggregate and cement content are factors that significantly affect cracking. This past research also found that high early strength cements and aluminous cements increased cracking, as did increased strength, superplasticizers, and silica fume. Prolonged curing and air entrainment reduced cracking.

The authors investigated the susceptibility of restrained concrete to cracking analytically using equations presented by ACI 209. Analyses found that two of the most important materials factors affecting cracking are the concrete modulus of elasticity and creep. Recently there has been a trend to use concretes with high compressive strengths that also have high elastic moduli and low creep. These concretes develop higher stresses from restrained shrinkage and thermal effects and are more prone to cracking.

**Testing**

The cracking tendency of 39 concrete mixtures was investigated using a restrained shrinkage test consisting of a 75-mm (3-in.) wide, 150-mm (6-in.) tall concrete ring cast around a 19-mm (¾-in.) thick section of steel tubing with an exterior diameter of 300 mm (12 in.). This steel ring provides about 70 percent restraint when a linear free-strain distribution for shrinkage is assumed through the concrete. The steel tubing was instrumented with electrical strain gages sampled every 30 minutes to detect first cracking, indicated by an abrupt loss of compressive strain in the steel tubing. The concrete rings were also visually examined for cracking. The effects of many factors such as water-to-cement ratio, cement content, aggregate size and type, superplasticizer, silica fume, set accelerators and retarders, air entrainment, cyclic temperature, evaporation rate, curing, and shrinkage-compensating cement concretes were investigated.

Except where noted, all mixtures used the same granitic gravel and natural river sand from Eau Claire, Wisconsin, and cement from the same supplier. All mixtures had the same mortar (cement + sand + water) volume, coarse aggregate proportions, and gradation except when the gradation was altered to investigate the effect of coarse aggregate type and size.

Along with the rings, corresponding compressive strength cylinders and free-shrinkage prisms were cast according to ASTM C192 and C127, respectively. Compressive strengths were measured at 7 and 28 days. All free-shrinkage specimens remained in the 22°C (72°F) 50 percent relative humidity environment for the duration of testing and were measured periodically. Ring strains were periodically analyzed, and detailed visual inspections were done after large strain changes occurred. Once a concrete ring cracked, the initial crack width was measured and the ring was monitored for at least one additional week. Then, the crack width was measured again, the ring was photographed, and the concrete was removed to allow the steel inner-ring to be cleaned for reuse.

The first group of tests investigated the effect of water-cement ratio and cement content on cracking. Eleven mixes were cast with cement contents of 280, 390, or 500 kg/m$^3$ (470, 658, or 846 lb/yd$^3$), and water-cement ratios of 0.30, 0.35, 0.40, 0.44, or 0.50. The cracking tendency of the concrete generally increased as the cement content increased and the water-cement ratio decreased. The mixes that performed best had essentially no slump, low cement contents, and low water-cement ratios but required excessive compaction for proper consolidation. Free-shrinkage was directly proportional to the paste content. The relationship between paste content and free-shrinkage is more distinct than that between paste content and the restrained ring cracking tendency. This reflects the difficulty in predicting cracking, which is affected by the complex interaction of restrained shrinkage, strength and modulus of elasticity development, and creep, factors not influencing free-shrinkage significantly.

Tests investigated the effect of shrinkage-compensating materials and mineral admixtures on cracking. Three concrete mixes were produced: one with a Type K cement, one with a commercially available expansive additive, and one with a 28-percent replacement of cement with a Type F fly ash. These mixes had cement contents of 390 kg/m$^3$ (658 lb/yd$^3$) and water-cementitious ratios of 0.44. The rings cast with Type K expansive cement cracked much later than the control concrete, and distinct wide cracks did not develop. The rings cast with the expansive additive cracked much later than the control mix, although a distinct crack eventually formed. The specimens with fly ash cracked slightly later than the control specimens.

Chemical admixtures, including HRWRAs, set accelerators and retarders, and air-entraining admixtures were investigated using seven mixes. The mixes had cement contents of 278 or 390 kg/m$^3$ (470 or 658 lb/yd$^3$), and water-cementitious ratios of 0.44 or 0.35. All mixes were compared to companion concretes with the same mix proportions.

Except for the no-slump concrete (278 kg/m$^3$ or 470 lb/yd$^3$, 0.35 water-cement ratio), the addition of HRWRA delayed cracking, even when silica fume was used. This contradicts reviewed literature. On average, concretes with accelerators or retarders cracked slightly sooner than control specimens, but high inter-ring variability precludes the drawing of any conclusions regarding the effect of these admixtures. Also, contrary to research by others, air entrainment did not significantly affect cracking.

Two mixes with silica fume were tested: one with a HRWRA and the other without. These mixes used a 7.5 percent addition of silica fume to the control mixes containing 390 kg/m$^3$ (658 lb/yd$^3$) of cement and a water-cement ratio of 0.35. The addition decreased the water-cementitious ratio to 0.33. The mixes containing silica fume cracked earlier than the companion mixes without silica fume.

Aggregate type had the most dramatic effects on cracking. The authors investigated four different aggregates. The aggregate types and sizes by ASTM C33, *Standard Specification for Concrete Aggregates*, were No. 8 (9.5-mm [⅜-in.] maximum size) lightweight-expanded shale, No. 56 (25 mm [1 in.]) crushed limestone, No. 8 trap rock, and No. 7 (12.5-mm [½-in.]) graded Eau Claire river gravel. These mixes were compared with a companion concrete using a cement content of 390 kg/m$^3$ (658 lb/yd$^3$) with a water-cement ratio of 0.44. The companion control concrete used a 19 mm (0.75 in.) maximum aggregate, classified as size No. 67.

The performance of the concrete rings made with No. 56 crushed limestone was unique. These rings did not develop any distinct cracks but showed many barely visible surface cracks around the perimeter. These cracks penetrated only about 25 mm (1 in.) into the concrete ring, did not extend to the steel ring, and could only be seen when alcohol was sprayed on the concrete ring. These steel rings underwent a slow gradual loss of compression strain. The rings were dismantled after approximately 280 days, after strains stabilized. This concrete had a moderately high modulus of elasticity of 34 GPa ($4.93 \times 10^6$ psi), similar to the control 35 GPa ($5.08 \times 10^6$ psi), but performed differently because it was crushed and had a slightly larger maximum size.

In contrast to the limestone rings, the lightweight-concrete rings developed large external cracks with no loss in steel ring strain. Only a change in the slope of the steel ring strain-time curve was noted, probably because of the extremely low modulus of elasticity of the lightweight concrete, 14 GPa ($2.07 \times 10^6$ psi), which limited shrinkage stresses. When the crack formed, the low stored energy was partially dissipated in cracking and absorbed through aggregate interlock across the crack, and by friction between the concrete and steel ring.

Specimens made with the hard trap rock aggregate had an elastic modulus similar to the control concrete, approximately 28 GPa ($4.08 \times 10^6$ psi), but cracked much later (32 days) than the control concrete (20 days). Another major difference between the aggregates was shape, with the trap rock being angular and the Eau Claire control concrete aggregate being well-rounded.

Curing duration was investigated using seven concrete mixes subjected to curing regimes including: no curing, 6-hr delayed curing, 60-day wet curing, and thermally insulated curing. The different curing conditions were tested on high- and low-cracking-tendency concrete mixes selected from the mixes used to investigate the effect of mix proportions on cracking. The low-cracking-tendency concrete had a cement content of 278 kg/m$^3$ (470 lb/yd$^3$) and a water-cement ratio of 0.50. The high-cracking-tendency concrete had a cement content of 500 kg/m$^3$ (846 lb/yd$^3$) and a water-cement ratio of 0.35.

The rings that were not cured (they were stripped out of the forms immediately after reaching final set) cracked quicker than the control specimens for both the high- and low-cracking-tendency mixes. No difference was noted for the 6-hr delayed curing concrete, although the geometry of the test specimens effectively prevented the elimination of evaporation from the concrete test surface during and before initial set.

Extending the wet curing to 60 days delayed cracking of the high-cracking-tendency mixes, but showed mixed results for the low-cracking-tendency mix. The effect of insulating the concrete after reaching peak-hydration temperature in order to slow heat loss was inconclusive.

The effect of evaporation rates was tested on the high- and low-cracking-tendency mixes described above. The testing was done by placing the concrete rings—24 hrs after casting—in an environment with an evaporation rate of 0.96 kg/m$^2$/hr (0.20 lb/ft$^2$/hr). The rings were cured in the forms under saturated burlap for 24 hrs before the forms were stripped. This precluded the development of plastic shrinkage cracks in the surface of the concrete. Rings placed in the high-evaporation-rate chamber cracked much earlier than companion rings placed in an environment with a lower evaporation rate of 0.15 kg/m$^2$/hr (0.03 lb/ft$^2$/hr). This result agrees with the findings of previous work, which linked the higher-evaporation shrinkage rates to earlier cracking. Strength development is not very dependent on the evaporation rate. Larger shrinkages cause larger shrinkage stresses, yet the concrete strength is similar or lower, causing earlier cracking.

The last group of tests investigated the effect of casting time on cracking tendency. One set of mixes was cast in a simulated morning placement; another set was cast in a simulated evening placement. Each set consisted of two rings, each of the low- and high-cracking-tendency mixes as previously described. The testing investigated the interaction between the concrete casting temperature and environment temperature. The concrete rings were cast at room temperature using room-temperature materials, and placed into the cyclic temperature chamber immediately after casting. The rings cast in simulated morning conditions cracked sooner than the rings cast in simulated evening; however, there was some scatter in the test results.

## CONCRETE MATERIAL FACTORS

Some concretes are more likely to crack than others. The concrete material properties and material-related mechanisms that lead to early cracking are ranked in Table 1. Most concrete material properties are collectively and simultaneously evaluated during the restrained ring cracking-tendency test.

### Paste Volume-Free Shrinkage

Free-shrinkage of concrete prisms was not related to when the restrained rings cracked. However, several transportation agencies believe that deck cracking is related to drying shrinkage. Drying shrinkage is largely caused by water losses from the concrete to the atmosphere. The amount of shrinkage depends on many different factors. The major factors affecting drying shrinkage are paste volume and quantity of water within the mix. Other factors that affect free-shrinkage are aggregate type and gradation, environmental conditions (temperature and relative humidity), cement type, and cement sulfate content. Deck thickness and form type affect the drying rate. The following reduce drying shrinkage in a given environment:

- Reduce the paste volume and the total amount of water in the concrete,
- Maximize the amount of aggregate,
- Use a Type II cement, and
- Use aggregate with low-shrinkage properties.

TABLE 1 Factors affecting cracking

| Factors | Effect | | | |
|---|---|---|---|---|
| | Major | Moderate | Minor | None |
| **Design** | | | | |
| Restraint | ✓ | | | |
| Continuous/simple span | | ✓ | | |
| Deck thickness | | ✓ | | |
| Girder type | | ✓ | | |
| Girder size | | ✓ | | |
| Alignment of top and bottom reinforcement bars | | ✓ | | |
| Form type | | | ✓ | |
| Concrete cover | | | ✓ | |
| Girder spacing | | | ✓ | |
| Quantity of reinforcement | | | ✓ | |
| Reinforcement bar sizes | | | ✓ | |
| Dead-load deflections during casting | | | ✓ | |
| Stud spacing | | | ✓ | |
| Span length | | | ✓ | |
| Bar type—epoxy coated | | | ✓ | |
| Skew | | | ✓ | |
| Traffic volume | | | | ✓ |
| Frequency of traffic-induced vibrations | | | | ✓ |
| **Materials** | | | | |
| Modulus of elasticity | ✓ | | | |
| Creep | ✓ | | | |
| Heat of hydration | ✓ | | | |
| Aggregate type | ✓ | | | |
| Cement content and type | ✓ | | | |
| Coefficient of thermal expansion | | ✓ | | |
| Paste volume—free shrinkage | | ✓ | | |
| Water-cement ratio | | ✓ | | |
| Shrinkage-compensating cement | | ✓ | | |
| Silica fume admixture | | ✓ | | |
| Early compressive strength | | | ✓ | |
| HRWRAs | | | ✓ | |
| Accelerating admixtures | | | ✓ | |
| Retarding admixtures | | | ✓ | |
| Aggregate size | | | ✓ | |
| Diffusivity | | | ✓ | |
| Poisson's ratio | | | ✓ | |
| Fly ash | | | | ✓ |
| Air content | | | | ✓ |
| Slump[†] | | | | ✓ |
| Water content | | | | ✓ |
| **Construction** | | | | |
| Weather | ✓ | | | |
| Time of casting | ✓ | | | |
| Curing period and method | | ✓ | | |
| Finishing procedures | | ✓ | | |
| Vibration of fresh concrete | | | ✓ | |
| Pour length and sequence | | | ✓ | |
| Reinforcement ties | | | | ✓ |
| Construction loads | | | | ✓ |
| Traffic-induced vibrations | | | | ✓ |
| Revolutions in concrete truck | | | | ✓ |

[†] within typical ranges

The longer periods of moist curing do not necessarily decrease the final drying shrinkage, but may reduce the shrinkage rate, especially for high-strength mixes. If concrete with a high paste volume and shrinkage is used, it is especially important to minimize the deck restraint and keep the early strength low by replacing cement with pozzolans or using cement designed for low early strength.

### Aggregate Type

Aggregate type was the most significant factor affecting when concrete cracked. The time-to-cracking of the Eau Claire river gravel concrete averaged 20.5 days, compared with 32 days for trap rock aggregate, 60 days for lightweight aggregate, and more than 280 days for limestone-aggregate concretes. The limestone aggregate was the only mix tested, except the mix containing Type K cement, that did not develop full-depth cracks in the restrained ring test.

To reduce cracking, Kosel (*17*) recommended a high ratio of aggregate to paste. Several researchers recommended large-sized aggregates, and a smooth grading curve to reduce bleeding (*6*).

To reduce deck cracking, Portland Cement Association (PCA) (*6*) recommended avoiding aggregates with high-shrinkage characteristics and selecting low-shrinkage aggregates. The type of aggregate also has a pronounced effect on crack length. Aggregate type and size influence the strength, elastic modulus, shrinkage, and creep—properties that greatly affect cracking. If high-shrinkage aggregates must be used, the batch properties should be optimized to reduce the cracking tendency. Mixes having low cement contents, shrinkage-compensating cement, low early strength gain, and low paste volumes should be tested. Reducing the restraint conditions of the deck will also help reduce cracking.

### Shrinkage-Compensating Cement

Shrinkage-compensating cement holds promise as a means to reduce transverse cracking, although concrete with this cement has varying field performance. Type K shrinkage-compensating cement prevented cracking of both of the restrained ring test specimens. The rings had very light surface cracking, but did not have a distinct crack or an abrupt reduction in steel ring strain. The ring strains gradually decreased to a constant level without cracking. Another shrinkage-compensating concrete containing an ettringite-forming expansive additive cracked at an average age of 36.5 days, 16 days later than the controls (20.2 days). However, because the ring test does not restrain the concrete but girders restrain deck expansion, the ring test may not accurately simulate field behavior. Supporting girders, reinforcing bars, and external abutments restrain the concrete deck as it tries to expand. If the initial expansion is greater than subsequent drying shrinkage, the concrete will remain in compression. Even if the drying shrinkage is greater, the combined stress is expected to be less.

Several transportation agencies found that the use of shrinkage-compensating cement reduced deck cracking (*9,18,19*). Researchers have written many reports on the use of shrinkage-compensating cements. Cusick (*20*) and Pfeifer (*21*) showed that shrinkage of shrinkage-compensating concrete is similar to that of normal portland cement concretes. Shrinkage decreased as restraint increased, but the decrease was slight when compared with the decreased expansion that occurred with the increased restraint. In nearly all cases, shrinkage of the shrinkage-compensating concrete was between 400 and 600 $\mu\epsilon$, despite a wide variation in initial expansion.

Hanson, Elstner, and Clore (*22*) investigated the expansive properties of Type M cement concretes in laboratory specimens that were totally restrained against either expansion or contraction. During the expansion phase, the specimens developed large compressive stresses that dissipated within 12 to 18 hrs. Cracking occurred in both Type I and Type M specimens during drying, but the specimens containing Type M cement cracked later than those with Type I cement. Although restrained concrete made with Type M cement initially develops compressive stresses, these stresses can dissipate quickly and cracking may occur.

Another study (*23*) on shrinkage-compensating concrete slabs found that these slabs had lower final net drying shrinkages than slabs made with Type I cement. The initial expansion of the reinforced slabs did not compensate completely for drying shrinkage. Lightly reinforced concrete slabs had much more expansion than heavily reinforced ones, but the amount of slab reinforcement had very little affect on subsequent shrinkage. Slabs containing shrinkage-compensating concretes crept more than corresponding slabs made with Type I cement.

Researchers studied the behavior of concrete made with Type II cement and shrinkage-compensating cement (*24*). In specimens with shrinkage-compensating cement, compressive stresses developed during curing. As the slabs dried, strain decreased and dropped below the initial value. Thus, tensile stresses formed in the concrete. An optimum percentage of steel of between 0.53 and 2.41 caused the highest compressive stresses. The slabs cracked early, and the researchers doubted the effectiveness of Type K cements to reduce cracking.

Researchers investigated five bridge decks constructed with shrinkage-compensating cement and adjacent structures with Type II cement (*25*). Shrinkage-compensating cement reduced cracking by approximately 25 percent. Some decks with shrinkage-compensating cement cracked more than those with Type II cement. The researchers concluded that to obtain significant benefit in areas subjected to deicing salts, crack reduction roughly of 90 percent is necessary, and the benefits of shrinkage-compensating cement they tested are not worthwhile.

One field survey examined more than 100 projects made with shrinkage-compensating cement concrete, including parking structures, slabs-on-ground, and miscellaneous installations (*26*). The study concluded that, on average, the shrinkage-compensating cement was very effective in reducing drying shrinkage cracks. Although the researchers gave high effectiveness rankings for slabs with as little as 0.05 percent reinforcement, better ratings typically occurred with higher percentages of steel.

The results of laboratory and field investigations concerning the use of shrinkage-compensating cement to reduce the incidence of early cracking are promising but mixed. The performance of concrete made with shrinkage-compensating cement appears variable, therefore further research on shrinkage-compensating cement in bridge deck structures is needed.

### Water-Cement Ratio and Cement Content

Many researchers observed more deck cracking with higher cement contents, such as 390 kg/m$^3$ (658 lb/yd$^3$) or more (*14,16,27*). They attributed this to the combination of higher heat gain during hydration, higher shrinkage, higher early modulus of elasticity, and lower creep. However, some researchers (*28*) found no relationship between the amount of cement and the extent of cracking. Some researchers recommend maximum cement contents of 370 kg/m$^3$ (620 lb/yd$^3$) (*27*) and 335 to 430 kg/m$^3$ (564 to 611 lb/yd$^3$) for deck concrete.

The average time-to-cracking for the different water-cement ratios and cement content mixes tested with this project's ring test are shown below.

| Cement factor | | Average time-to-cracking (days) water-cement ratio | | | | |
|---|---|---|---|---|---|---|
| kg/m$^3$ | (lb/yd$^3$) | 0.30 | 0.35 | 0.40 | 0.44 | 0.50 |
| 280 | 470 | 40.5 | 53.0 | — | 19.8 | 25.0 |
| 390 | 658 | — | 17.6 | 17.0 | 20.2 | 18.5 |
| 500 | 846 | 10.5 | 11.7 | — | 20.1 | — |

The best time-to-cracking results were achieved with the low cement factor mixtures of 280 kg/m$^3$ (470 lb/yd$^3$) with no-slump concretes at 0.30 and 0.35 water-cement ratios. These two no-slump concretes were very dry and not practical for bridge decks. Both mixes contained very low paste volumes because the water and cement contents were low. When a HRWRA was added to the mix with a water-cement ratio of 0.35 and 278 kg/m$^3$ (470 lb/yd$^3$) of cement, the slump was increased to 127 mm (5 in.) and the average time-to-cracking was 25.5 days, compared to 53.0 days for the same mix that had no slump. Therefore, the excellent performance of the no-slump mixes is related to their dry consistency and not their mix proportions.

The other nine mixtures had slumps ranging from 25 to 250 mm (1 to 10 in.), and these mixtures cracked at ages ranging from 10 to 25 days. All concrete specimens with a cement content of 390 kg/m$^3$ (658 lb/yd$^3$) cracked at essentially the same age (between 17 and 20 days), yet slump ranged from 37 to 254 mm (1½ to 10 in.) and water contents ranged from 137 to 195 kg/m$^3$ (230 to 329 lbs/yd$^3$). These data suggest that cracking of these typical, moderately high cement-content mixes was not dramatically affected by water content or water-cement ratio from 0.35 to 0.50.

All three AASHTO-quality 0.44 water-cement ratio mixtures with three cement contents from 280 to 500 kg/m$^3$ (470 to 846 lb/yd$^3$) cracked at an age of 20 days, although their water contents ranged from 123 to 220 kg/m$^3$ (207 to 372 lb/yd$^3$) and their slumps ranged from 25 to 254 mm (1 to 10 in.).

The data for the 0.30, 0.35, and 0.50 water-cement ratio mixtures show that increasing the cement content causes quicker cracking.

These data suggest that no-slump, low cement content, low water-cement ratio mixes cracked latest; high cement content, low water-cement ratio mixtures with slump qualities cracked earliest; and concretes with typical and moderate 0.40 to 0.50 water-cement ratios and typical cement contents produced intermediate time-to-cracking characteristics. These data seem to parallel the common perception of increased cracking relating to the recent use of higher cement content and lower water-cement ratio concretes.

The data also suggest that the slump of low water-cement ratio concrete does not play a significant role in time-to-cracking, because the high cement content mixtures with 0.30 and 0.35 water-cement ratios cracked at 11 days, yet one mixture had a 50-mm (2-in.) slump and the other had a 254-mm (10-in.) slump.

A slight statistical relationship between water-cement ratio ($R^2 = 0.50$) and age at cracking was seen. The two no-slump mixes were not included in the regression analysis. Concretes with high cement contents and low water-cement ratios are more susceptible to cracking than concretes with low cement contents and high water-cement ratios.

### Concrete Strength

There has been a significant trend toward increasing the 28-day design strengths of concrete. In addition, the lower water-cement ratio concretes being specified frequently for corrosion protection produce 28-day compressive strengths far above the design compressive strengths. In fact, some specified mixtures can achieve 28-day design strengths in 3 to 7 days. Many agencies have suggested that this trend has led to more cracked structures. The primary reasons for increased cracking are increased cement contents, higher paste volume, higher early modulus of elasticity, higher hydration temperatures, and much lower creep.

A slight statistical relationship was noted ($R^2 = 0.63$) between the 7-day compressive strength and cracking tendency for the restrained ring specimen mixes having varying cement contents and water-cement ratios. The no-slump 278 kg/m$^3$ (470 lb/yd$^3$) mixes with water-cement ratios of 0.30 and 0.35 were excluded.

### Water Content

Horn (*29*) found little correlation between cracking and the water content in concrete decks. However, other researchers have suggested that reducing the water content will reduce deck cracking (*9,30*). This project did not find a relationship between total mix water and when the restrained rings cracked. Changing the total water from 122 to 220 kg/m$^3$ (207 to 372 lb/yd$^3$) did not affect the age at cracking for the AASHTO-quality 0.44 water-cement ratio mixtures. The 0.30, 0.35, and 0.50 water-cement ratio mixtures cracked sooner as water contents increased; however, these mixtures also had increased cement contents and paste volumes. The water content and paste content are primary factors affecting shrinkage and creep. Concrete with more water shrinks and creeps more than concrete with less water, but it may not crack sooner because it has higher creep.

### Air Content

The restrained rings cast with air entrainment (4.5 percent) did not show a cracking tendency significantly different from the non-air-entrained concrete (2.2 percent). However, other research suggests that increasing air content decreases cracking (*14,31*). Air content is usually specified for freeze-thaw durability requirements; however, it may be advantageous to use air-entrained concrete in environments that are not subject to freezing and thawing cycles.

### Slump

This project's results indicate there is no relationship between slump and cracking tendency with the restrained ring tests. Five mixtures with slump from 0 to 50-mm (0 to 2-in.) cracked at ages of 11 to 53 days, while two mixtures with 100- to 190-mm (4- to 7½-in.) slump cracked at ages of 17 to 25 days, and four mixtures with 215- to 254-mm (8½- to 10-in.) slump cracked at ages of 12 to 19 days. Horn, Stewart and Boulware (*28*) also found that slump did not affect deck cracking.

Several researchers suggested that high slump concretes are more susceptible to settlement cracking than those with lower slumps (*6,17,30*). However, settlement cracking is the result of poor construction practice and inadequate vibration. Excessively high slumps should be avoided, but within typical ranges, slump does not affect cracking. Although impractical for use, the concrete rings made with no-slump, low cement content, and low water-cement ratio mixes cracked latest.

### Modulus of Elasticity

This project's analytical studies showed that the concrete modulus of elasticity, adjusted for creep, affects both thermal and shrinkage stresses more than any other physical concrete property, despite girder type or bridge geometry. Increasing the concrete modulus of elasticity increases both shrinkage and thermal stresses. The modulus of elasticity and the associated creep are important because their interaction determines stress for a given strain.

The concrete modulus of elasticity is largely affected by the modulus of elasticity of the aggregates, which comprise most of the concrete matrix. Concretes with less stiff aggregates typically have lower moduli of elasticity and higher creep. Using low-elasticity aggregates should therefore reduce thermal and shrinkage stresses, and the risk or severity of transverse cracking.

The modulus of elasticity increases with compressive strength, as does tensile strength. The tensile strength and modulus of elasticity are commonly calculated as proportional to the square root of the compressive strength of the concrete.

As an example of how compressive strength can affect cracking, consider two concretes with respective compressive strengths of 25.8 and 51.7 MPa (3750 and 7500 psi). These two concretes have ACI-calculated moduli of elasticity values of about 24.0 and 35.4 GPa (3.5 and 5.0 $\times$ 10$^6$ psi), respectively. Typical long-term creep factors for these two concretes can be about 3.4 and 1.25 times the elastic strains, respectively, and their flexural tension cracking strengths would be about 3.2 and 4.5 MPa (460 and 650 psi), respectively. Calculations indicate that these two concretes, when totally restrained, can tolerate about 590 and 290 $\mu\epsilon$, respectively, after accounting for their actual tensile strengths, moduli of elasticity, and tensile-creep properties. Therefore, increasing the compressive strength 100 percent increased the tensile strength and the modulus of elasticity 42 percent, but decreased the creep 75 percent, resulting in a concrete that will crack at only 50 percent of the strain at which the lower strength concrete will crack. These calculations suggest that the concrete modulus of elasticity and creep dominate cracking-tendency, and that high-strength, low-creep concretes are more prone to shrinkage cracking.

Reducing the concrete modulus of elasticity reduces shrinkage and thermal stresses of concrete. Using low modulus aggregates, increasing the paste content, and using lower strength pastes (and concretes) reduces concrete modulus of elasticity. However, these same factors can increase total shrinkage.

The restrained ring test evaluated five AASHTO-quality 0.44 water-cement ratio, normal-weight aggregate mixtures, with modulus of elasticity values from 27.6 to 34.5 GPa (4 to 5 $\times$ 10$^6$ psi) and one AASHTO-quality lightweight-aggregate mixture with a modulus of 14.7 GPa (2.1 $\times$ 10$^6$ psi). Five of the six mixtures cracked at an age between 20 and 30 days. The other, the concrete with limestone aggregate, developed many narrow and short surface cracks, less than 25-mm (1-in.) deep; it did not develop distinct wide cracks. This concrete exhibited a moderately high modulus of 34.0 GPa (4.9 $\times$ 10$^6$ psi). These data suggest that, in addition to modulus of elasticity, aggregate type or shape also plays a dominant role in deck cracking.

### Creep

As discussed, concrete creep reduces tensile stresses from restrained drying shrinkage and thermal effects, and reduces deck cracking. Thus, concrete with high creep, particularly during the first month after casting, is desired. Such a concrete with early high creep will have relatively low compressive strength and slow strength development. Conversely, concretes with high or very high early strength development will produce a low-creep concrete with much greater risk of cracking. Creep largely depends on when stresses are applied, the moisture conditions, and the aggregate properties.

Concrete made with very low water-cement ratios, high cement contents, silica fume admixtures, and other ingredients that produce very high early strengths and moduli of elasticity are prone to cracking because they creep little. Concretes cured by intentional heat-curing techniques at the jobsite or by unintentional solar radiation effects develop higher early strengths with lower creep and higher cracking risks.

Creep may occur either with or without drying shrinkage effects. Basic creep occurs without air drying, and drying creep is creep that occurs in addition to basic creep. Bridge decks commonly air-dry from both sides and drying creep dominates basic creep. Drying creep is typically 2 to 3 times basic creep when the air relative humidity is 70 to 50 percent, respectively. Decks with SIP steel forms dry only from one side, and basic creep and drying creep may occur in similar amounts during the early curing and drying periods.

High creep is most easily produced by using a high water-cement ratio concrete with low strength, soft aggregates, and poorly graded mixtures. However, such concretes also have high shrinkage, high chloride permeability, low abrasion resistance, and other durability concerns. Apparently, high creep can also be produced by designing and curing low water-cementitious ratio mixtures with very slow heat of hydration rates and by using intentional cooling to slow the early strength gain, allowing high creep during the critical first month. These slowly developing mixtures, often containing pozzolans, need longer moist curing. They can have low permeability, high strength, and high abrasion resistance, while having higher early creep, and low long-term shrinkage. To encourage slower strength gain, it may be advantageous to design bridge decks for 90-day compressive strengths instead of the usual 28-day strength.

### Heat of Hydration

Reducing placement and peak concrete temperatures relative to ambient air temperatures can reduce deck cracking (*9,27*). High temperatures can create thermal stresses that create early cracks, particularly during the subsequent rapid cooling. The temperature gained by concrete during hydration depends on the type and amount of cement, percentage of fly ash, aggregate-to-paste ratio, batching temperature, ambient environment, and solar radiation. Temperature changes induced internally by the hydration of portland cement and fly ash significantly affect young concrete. When large volumes of concrete are placed in areas that do not permit adequate dissipation of hydration heat, the concrete temperature may increase as much as 56°C (100°F). A rule of thumb is that a potential temperature rise of 7 to 8 °C (13 to 15°F) occurs per 45 kg (100 lb) of portland cement per 0.76 $m^3$ (1 $yd^3$) of concrete, starting at moderate construction temperatures.

Modern concretes and mortars having high 1-day strength would also be expected to have higher heats of hydration. Few data are available about the heat of hydration of modern portland cements because this test is no longer routinely performed. Modern finer cements producing high early strengths may result in hydration heat for Type II cements of 75 calories per gram compared to the 60 calories per gram typical in the late 1940s. This will aggravate cracking because the concrete will reach higher temperatures and result in more locked-in deck stress as the concrete cools to ambient. The stresses are further aggravated because the concrete has such a high early modulus.

To prevent excessive thermal gradients within the concrete, the concrete should have acceptable peak and placement temperatures; however, the transportation agencies do not agree on appropriate placement and peak temperatures. PCA (*6*) recommended a maximum concrete temperature at placement of 28°C (80°F). Several others recommended that a retarding agent be used to reduce the temperature rise (*27*). However, immediate and proper wet curing is essential for concretes containing retarders because they are susceptible to plastic cracking, particularly in hot or cold weather. Researchers also recommended cooling aggregates and mix water (*9,27*). The use of portland cements that have low heats of hydration and the use of low-permeability fly ash-blast furnace slag-portland cement mixtures can produce even more desirable early low heat of hydration.

### Concrete Coefficient of Thermal Expansion

Another factor that affects thermal stress in the concrete deck, with either steel or concrete girders, is the coefficient of thermal expansion of the concrete. The stresses that develop from a temperature change in the deck are linearly proportional to the concrete coefficient of thermal expansion.

Thermal stresses and transverse cracking can be reduced by using concretes with lower coefficients of thermal expansion. Increasing the aggregate content can reduce the concrete coefficient of thermal expansion by reducing the more thermally expansive paste content, and increasing the less thermally expansive aggregate. Using aggregates with lower thermal expansion rates also decreases the thermal expansion rate of the concrete. For example, concretes with limestone aggregate typically have less thermal expansion than concretes with quartzite aggregate. The coefficient of thermal expansion of cement paste (*32*) (with water-cement ratios of 0.4 to 0.6) is typically between 18 to 20 με/°C (10 to 11 με/°F), about 2 to 3 times that of aggregate. Researchers have reported the following aggregate expansion rates (*32*).

| Aggregate | Coefficient of thermal expansion με/°C (με/°F) |
|---|---|
| Granite | 7-9 (4-5) |
| Basalt | 6-8 (3.3-4.4) |
| Limestone | 6 (3.3) |
| Dolomite | 7-10 (4-5.5) |
| Sandstone | 11-12 (6.1-6.7) |
| Quartzite | 11-13 (6.1-7.2) |

Larger thermal stresses develop in a concrete deck when the temperature change is limited to the deck (either uniform or linear), and smaller stresses develop when the girders also change temperature. Seasonal (uniform full-depth) temperature changes typically cause very small thermal stresses in a concrete bridge deck. When steel girders support the deck, selecting a concrete with a high coefficient similar to steel could reduce stresses caused by a uniform temperature change in the deck and girders; however, this will increase the larger diurnal thermal stresses. When concrete girders support the deck, the thermal stresses from seasonal temperature changes are generally insignificant when compared with diurnal temperature changes and drying shrinkage.

### Cement Type

Other researchers found that type of cement has a large effect on deck cracking (*28,35*). Some (*28*) found that decks constructed with Type II cement cracked less than those constructed with Type I cement, and recommend Type II cement to reduce early thermal gradients and shrinkage (*17,27*). Type III cement gains strength rapidly and may increase the risk of cracking. Certain brands of cement have higher shrinkage than others. No information on the effect of Type IV cements, which have lower thermal gains than other cements, was obtained in this project.

The general chemistry and fineness of cements have changed during the last 20 years. In the mid-1970s, the AASHTO Specifications increased the minimum concrete compressive strengths for decks from 20 to 31 MPa (3000 to 4500 psi). Contractors also sought cements that achieve quicker strengths to speed form removal and access to the deck. In response, cement producers changed the cement fineness (Blaine) and composition. The most expensive process and the highest cost associated with cement manufacturing is grinding. Beginning in 1970, cements were ground finer, resulting in cements with higher sulfate and alkali contents. The increased sulfate content was intended to control the faster (finer) reacting aluminates, and the increased alkali content resulted from the use of inexpensive fuel types to heat the kiln, such as trash and old tires, and pollution control requirements.

The finer cements and higher sulfate contents increased early strengths, heats of hydration, and the early modulus of elasticity. For example, the deck concrete on the Portland-Columbia Bridge had a 1-day modulus of elasticity of 19,200 MPa (2.8 × $10^6$ psi), nearly 80 percent of its ultimate modulus. Lower-strength concretes produced before 1970 typically had 2-day elastic moduli of only 40 percent of their 28-day value. The modulus of elasticity directly affects the stresses that cause transverse deck cracking.

From 1904 to 1941, cement strength was mainly controlled and tested in direct tensile. After 1941, minimum compressive strengths were specified. From 1904 to 1941, the tensile-strength requirements in ASTM Specifications increased from 1.0 MPa (150 psi) to 1.9 MPa (275 psi) at 7 days and 1.4 MPa (200 psi) to 2.4 MPa (350 psi) at 28 days. The ASTM C150 Specifications for Type I portland cement since 1941 had the following minimum limits for compressive strength of mortars.

| Type I cement specification (ASTM C150) | Compressive strength of mortar | | |
|---|---|---|---|
| | 3-day | 7-day | 28-day |
| Year | MPa (psi) | MPa (psi) | MPa (psi) |
| 1941 | 6.9 (1000) | 13.8 (200) | 20.7 (3000) |
| 1947 | 2.6 (900) | 12.4 (1800) | 20.7 (3000) |
| 1955 | 8.3 (1200) | 14.5 (2100) | 24.1 (3500) |
| 1974 | 12.4 (1800) | 17.3 (2800) | 24.1 (3500) |
| 1976 | 12.4 (1800) | 19.3 (2800) | 27.6 (4000) |
| 1989 | 12.4 (1800) | 17.3 (2800) | — — |
| 1994 | 12.4 (1800) | 9.3 (2800) | — — |

The specification changes in 1947 and 1955 parallel changes in the method of test. In 1947, the last requirement for a fixed 0.53 water-cement ratio was eliminated. In 1955, the mortar mixing procedure was changed from hand mixing to machine mixing. The significant increases in early age C150 mortar strength requirements in the mid-1970s correspond to the strength increases in the AASHTO deck concrete specifications and to the time when deck cracking is believed to have increased.

The 1-day compressive strengths for the Portland-Columbia Bridge deck concrete were 53 to 95 percent of the 28-day strength. Compare this to the following data presented in Table 3-10 of the PCA Bulletin 26 (*33*) on mortar cubes published in 1948.

| | Compressive strength | | | | |
|---|---|---|---|---|---|
| | 1-day | 3-day | 7-day | 28-day | 1-day percent of 28-day |
| Type | MPa (psi) | MPa (psi) | MPa (psi) | MPa (psi) | |
| I | 3.6 (538) | 11.6 (1689) | 20.3 (2944) | 33.0 (4789) | 11 |
| II | 2.8 (412) | 8.1 (1180) | 14.0 (2034) | 25.7 (3728) | 11 |
| III | 10.0 (1457) | 23.8 (3457) | 35.2 (5103) | 43.1 (6250) | 23 |
| IV | 2.0 (288) | 4.7 (688) | 6.9 (1000) | 17.3 (2510)* | 11 |
| V | 2.7 (400) | 8.5 (1240) | 12.3 (1780) | 20.8 (3010)** | 13 |

* 3-month strength was 35.2 MPa (5110 psi)
**3-month strength was 38.3 MPa (5550 psi)

Compressive strength tests on 152 mm by 305 mm (6 in. by 12 in.) concrete cylinders using cements manufactured in the 1940s were reported by Gonnerman and Lerch (*34*). Type I cement concretes had average 1-day compressive strengths of 5.4 MPa (776 psi); only 14 percent of the 28-day strength of 39.1 MPa (5676 psi). Type II cement concretes had average 1-day compressive strengths of 3.0 MPa (430 psi); only 12 percent of the 29-day strength of 24.5 MPa (3556 psi).

The 1-day strengths of these early cement mortars and concretes were only 11 to 14 percent of the 28-day strength compared to modern cement concretes that obtain 40 to 60 percent of the 28-day strength within the first day. Modern concretes with such high early modulus and compressive strength values dramatically increase the risk of cracking because of the high stresses that develop as a result of early shrinkage and thermal strains.

## Mineral Admixtures

Many transportation agencies use mineral admixtures. The main two mineral admixtures used in the United States are fly ash and silica fume.

**Fly Ash.** Fly ash is being used in increasing quantities. Fly ash, especially Class F and Class N, reduces the rate of strength gain and early concrete temperatures, and has been recommended to reduce deck cracking (*27*). Replacing 28 percent of the portland cement with Type F fly ash did not significantly affect when the concrete rings cracked.

**Silica Fume.** Investigators (*36,37*) have blamed silica fume for bridge deck cracking, but this issue is controversial. Silica fume concrete typically reaches higher temperatures during early hydration, which causes larger thermal stresses. Silica fume concrete also usually bleeds much less than normal concrete and is more prone to plastic shrinkage cracking.

Researchers (*38*) found that very high-strength silica fume concrete undergoes intense autogenous shrinkage because of its extremely low water-cement ratio. They noted that conventional concrete shrinks very slowly after the initial stage of hydration swelling. Silica fume concrete did not swell during hydration but instead shrank immediately. They attributed this high autogenous shrinkage of silica fume concrete to self-desiccation.

Concrete laboratory rings containing 7.5 percent additional silica fume cracked 5 to 6 days sooner than control concretes without silica fume. The cracking tendency of silica fume mixes was higher, though the free-shrinkage was similar to the control. The earlier cracking may be related to the higher elastic modulus and lower creep of the silica fume concrete, which causes higher stress for a given strain.

## Retarders

Some investigators have blamed set retarders for deck cracking, but others found no relationship (*8*). Concrete rings with a mix containing ASTM C494 Type B/D retarder had a final set time of 9.7 hrs, and on average cracked 2 days sooner than the control mix; however, the results were scattered, which precludes definite conclusions. Retarders are often used to enable a concrete deck to be cast continuously. Retarders slow the early tensile strength gain and increase susceptibility to plastic cracking; however, retarders reduce temperature gain during early hydration, which should reduce the risk of thermal cracking. When concrete set is retarded, it would be necessary to analyze changes in dead-load deflection of the plastic concrete during placement to prevent cracking. Evaporation retarder films or fogging should reduce the risk of plastic shrinkage cracks when retarders are used in hot or cold weather.

### Accelerators

The effect of accelerators on transverse deck cracking is generally unknown. Concrete rings containing an accelerator cracked slightly sooner (4 days) than the control mix in the cracking-tendency ring tests. Accelerators are rarely used in the construction of concrete decks. However, certain admixtures intended for other purposes, such as corrosion inhibiting, may accelerate setting. Accelerators increase the strength gain and the stresses necessary to cause cracking. Sometimes, the rapid strength gain may be advantageous and reduce plastic cracking. However, accelerators can increase shrinkage, early temperature rise, and early modulus of elasticity, all of which aggravate cracking.

### Concrete Thermal Diffusivity

Concrete thermal diffusivity is a measure of how readily heat flows through concrete; a larger value indicates quicker heat conduction. The thermal analyses of steel girder and concrete girder bridges revealed that nonuniform temperature changes produce larger thermal stresses than uniform temperature changes. Concrete decks with higher diffusivity will have smaller temperature gradients than decks with lower diffusivity, and hence lower thermal stresses. For example, concrete decks constructed with basalt aggregate are expected to have larger hydration and diurnal temperature gradients and stresses than decks constructed with more conductive quartzite aggregate. ACI 207.1R (*39*) lists the following diffusivity values for concretes of various aggregate types.

| Coarse aggregate | Diffusivity of concrete $m^2$/day ($ft^2$/day) |
|---|---|
| Quartzite | 0.129 (1.39) |
| Limestone | 0.113 (1.22) |
| Dolomite | 0.111 (1.20) |
| Granite | 0.096 (1.03) |
| Rhyolite | 0.078 (0.84) |
| Basalt | 0.072 (0.77) |

### Aggregate Size

Kosel (*17*) recommended a minimum aggregate size of 25 mm (1 in.), but Carlson (*14*) found no correlation with cracking and aggregate size. Cracking-tendency tests of concretes containing Eau Claire aggregate with maximum sizes of 19 mm (¾ in.) and 13 mm (½ in.) showed no significant difference.

### Poisson's Ratio

Poisson's ratio typically has a small effect on shrinkage and thermal stresses in the deck of a bridge. Deck stresses generally increase as Poisson's ratio increases, so selecting a concrete with a lower Poisson's ratio will generally reduce shrinkage and thermal stresses that may cause transverse cracking. Poisson's ratio typically ranges from 0.15 to 0.20, with higher strength concretes typically having higher ratios. Therefore, reducing the compressive strength of the concrete often reduces the strains and stresses that develop in the deck from Poisson's effect, and may reduce the risk or severity of transverse cracking.

### Fiber Reinforcement

One researcher suggested that fiber reinforcement reduces early plastic cracking (*9*). In Japan, steel fibers are used to reduce deck cracking. Some researchers found that fibers (1) reduce plastic and settlement cracking and (2) reduce crack width (*15*).

### Summary of Concrete Material Influences and Recommendations

Many concrete material properties affect the susceptibility of a concrete deck to cracking. Table 1 lists the various material factors and ranks them in order of importance. Recommendations on material properties to reduce cracking include the concretes with a low cracking tendency, i.e., concretes with the following properties:

- Low early modulus of elasticity,
- Low early strength concrete (use 60- or 90-day design strengths),
- High early creep,
- Low amounts of portland cement,
- Good quality, low-shrinkage aggregates,
- Low hydration temperatures (by using cements with low hydration heat),
- Low heat of hydration pozzolans,
- Low thermal coefficient of expansion,
- Minimum paste volumes and free-shrinkage,
- Type II cement, and
- Shrinkage-compensating cement.

Also, use of finely ground cement, silica fume, and other admixtures that produce very high early and later strengths and moduli of elasticity should be avoided.

Air entrainment, water reducers, retarders, and accelerators have minimal effects on cracking. The ring tests indicate that aggregate size, slump, and water content do not significantly influence cracking. Potential trial mixes can be evaluated with the ring test to identify those mixes with low cracking tendency.

## DESIGN FACTORS

Design details can affect deck cracking and are of particular interest because design changes can be readily implemented. Table 1 also ranks the design factors, and a discussion of each factor follows.

### Girder Restraint

A concrete deck of a composite bridge is typically restrained externally only at the girder interface. Transverse deck cracking would be eliminated or greatly reduced without this restraint. Unfortunately, isolating the deck is not economically practical for structural considerations, although many noncomposite decks were built in the past.

Researchers have proposed many methods of predicting crack widths for concrete in direct tension or pure bending; less has been done to predict cracking in base-restrained members subjected to both axial tension and bending. Unlike members restrained only at longitudinal ends, nor like beams in bending, the restraint adjacent to a crack in base-restrained concrete is not approximately zero. Instead, a shear force develops between the concrete and the base. As a result, less bond-slip occurs at the fracture, and the tensile force in reinforcing steel will redevelop in shorter lengths. Hence, cracks can occur at closer spacings (*40*).

In base-restrained walls, cracking begins at the base near mid-length where maximum restraint occurs, and propagates toward the top. With time, additional vertical cracks form toward the ends of the wall. This is representative of uniform shrinkage or contraction through the depth of the deck. However, actual strain is likely to be nonuniform because of uneven drying shrinkage or nonuniform temperature gradients.

Equations were derived analytically to allow estimates of degree of restraint provided by different bridge systems and concrete materials. These equations allow designers to select bridge systems with minimum degree of restraint to help eliminate deck cracking problems.

### Continuous and Simple Spans

Cracking is more prevalent on continuous spans than on simple spans; however, cracking occurs on both types of structures. Transportation agencies have observed more cracking in the middle spans of continuous structures than in the end spans. One study found cracking more prevalent on two-span systems than on other continuous-span systems (*5*).

### Concrete Strength

Concrete strength is a material property considered in structural design. Concrete with high early strength generally has a high cement content and is prone to cracking. This is due to the high modulus of elasticity and low creep of high-strength concretes. Bridge decks should not be built with unusually high-strength concrete, and 56- or 90-day design strengths should be considered instead of 28-day strengths.

### Temperature Changes

Investigators have blamed temperature changes for early deck cracking (*36*). Temperature changes are not usually considered in design because temperature steel is usually considered sufficient to control cracking, a practice that apparently is not working satisfactorily.

Bridges are continuously subjected to changing temperatures; therefore, a significant factor in the behavior and performance of bridges is thermal loading. AASHTO design guidelines provide provisions only for longitudinal expansion and contraction of bridges caused by a uniform temperature distribution. However, temperature gradients occur in all bridges, and at all ages, and significant tensile stresses can develop, as discussed in this study.

Temperatures are rarely uniform in a bridge. A critical early temperature change occurs during the first day or two after the concrete is placed when the cement hydrates rapidly and generates heat. Cooling then follows this rapid heat gain. As the deck cools, it shrinks, but it is restrained by the girders. Changing early weather conditions such as air temperatures, solar radiation, wind, precipitation, and other factors also affect bridge temperatures. The early temperature drop produces large tensile stresses that contribute to transverse deck cracking.

When a deck is cast monolithically with concrete girders, thermal stresses caused by hydration are generally reduced because both the deck and girders generate hydration heat and then cool at the same time, and temperature differences are reduced. However, stresses still develop in this bridge from hydration, because sections of the deck cool quicker than the thicker girders.

Thermal stresses in the concrete from hydration are usually worse in steel-girder bridges. Concrete girders conduct heat slower than steel ones, and the greater mass of the girders will cause them to respond slower to the changing deck temperatures. Steel girders typically conduct heat quicker than concrete girders, and upper flanges will heat and cool with the deck, reducing the temperature difference at the interface.

For most bridges, diurnal temperature changes produce the largest thermal stresses. The diurnal temperature cycle of a bridge deck usually exceeds the ambient air temperature cycle, especially when the deck is directly exposed to solar radiation. Bridge decks in moderate or extreme climates often experience 22°C (40°F) diurnal temperature cycles. Large girders, especially concrete girders, have large thermal masses and react more slowly to the changing environment; they often have smaller temperature cycles than ambient air. The upper surface of the deck typically heats and cools more quickly, because it is exposed to direct solar radiation and precipitation. Because heat does not transfer instantly to the girders, temperatures are rarely, if ever, uniform in a bridge. The parameter study from this project revealed that a linear rather than a uniform temperature gradient in the deck typically produces the largest deck stresses and greatest risk of transverse cracking.

Seasonal temperature changes produce small or negligible stresses in concrete girder bridges because both deck and girders typically have similar thermal expansion rates. Deck stresses in concrete bridges caused by seasonal (uniform full-depth) temperature changes are minor and only occur because of the expansion difference of the deck reinforcing steel.

Protecting the concrete from solar radiation to reduce the temperatures due to hydration and insulating the bridge to reduce the rate of cooling should reduce the incidence of early cracking.

### Effects of Girder Type

Structures supported on wide flange beams and composite steel-plate girders exhibited much more cracking than those constructed on other systems (*5*). It is believed that the increased cracking of steel girder structures is due to the stiffness and the thermal properties of steel compared with those of concrete girders.

The New Mexico DOT found that both prestressed concrete girders and steel girders can cause severe cracking. Cady (*8*) found cracking worse with precast, prestressed girders when the deck was conventionally formed, and worse with steel girders when SIP forms were used. The South Dakota DOT found cracking worst on prestressed concrete beams and attributed this to a greater bond between the deck and the girder. Analytical studies for this project indicate that steel girders usually cause higher thermal and shrinkage stresses in simply supported spans, but that concrete girders often cause higher stresses when spans are continuous over supports; this may explain the inconsistent survey responses.

### Dead-Load Deflections

Some researchers have blamed excessive dead-load deflections for transverse deck cracking, while others could not relate dead-load deflections to cracking (*8*). Transverse cracking of the plastic concrete can occur over the supports of continuous unshored structures as a result of self-weight bending. Cracks developed between 2 and 4.5 hrs after mixing when a researcher applied a curvature of 0.02 $m^{-1}$ (0.0005 $in.^{-1}$) to concrete placed in flexible formwork simulating deck slabs (*41*). To prevent this type of plastic concrete cracking, falsework deflection should be calculated, and the construction sequence selected to eliminate the tensile stresses caused by self-weight.

### Stud Spacing

Very little information is published on the effect of stud spacing or other connections on cracking, and no relationship of cracking to restraint was established. The analytical studies in this project indicate that the restraint provided by steel studs and channels increases stresses locally by as much as 20 percent at the studs and may increase localized deck cracking.

### Deck Geometry

Geometry affects shrinkage and thermal stresses less than material properties do. Generally, stresses are larger in thin decks on deep girders at a narrower spacing than in thick decks on shallow girders at wide spacing. However, the interaction of geometry and material is complex, and many exceptions occur. Deck reinforcement has a small influence on thermal stresses in the deck.

Transportation agencies generally believe that longer spans crack more frequently than shorter spans (*5*,*8*). Pennsylvania stated that spans longer than 30 m (90 ft) are likely to crack. Data (*5*) suggest that decks wider than 21 m (70 ft.) are more susceptible to cracking than narrower decks. These beliefs are consistent with this project's analyses, which indicate that larger girders (required for longer spans) often produce higher shrinkage and thermal stresses in the deck.

### Concrete Cover

Analytical studies in this project showed that the depth of the reinforcement has an inconsistent influence on deck stresses. Some researchers (*4*,*5*) found deck cracking worse when the concrete cover was more than 75 mm (3 in.), but others (*28*,*42*) found no correlation. Spans with small cover are more susceptible to settlement cracking (*43*), and the probability of settlement cracking decreases as clear cover increases (*31*,*43*). As the reinforcing bars move away from the concrete surface, they become less effective at distributing tensile stresses. Cover from 38 mm (1.5 in.) to 75 mm (3 in.) is generally recommended (*5*).

### Quantity of Reinforcement

The analytical studies found that the amount and location of longitudinal deck reinforcement typically have a small effect on deck stresses. More reinforcement increases deck stresses and possibly cracking, but the additional stresses are usually small or negligible. Additional longitudinal reinforcement, especially smaller bars at a narrower spacing, will reduce transverse crack widths and improve serviceability.

Additional longitudinal reinforcement, $220/\sqrt{s}$ versus $100/\sqrt{s}$, as recommended by AASHTO, has successfully reduced leaking cracks (*44*). This higher level of reinforcement is typical of decks designed today. Temperature steel should be placed over the primary slab reinforcement.

### Deck Thickness

The analytical studies show that increasing the deck thickness usually decreases the shrinkage and the thermal stresses that develop in the concrete deck of a bridge. The effect is not always consistent, however, because the interaction with the girders is complex. Uniform shrinkage or temperature changes in the deck usually develop nearly uniform stresses in thin decks. On the other hand, thicker decks on smaller girders may develop significant bending stresses because of the distance of the deck centroid from the girder. Thicker decks are also more prone to develop nonuniform shrinkages and temperatures than thinner decks, which may also increase stresses. When high-strength concrete (high modulus of elasticity and Poisson's ratio) is used, especially with large steel girders, increasing the deck thickness usually decreases deck stresses, but its effect on deck stresses is often inconsistent with other geometries and concrete properties.

Current literature suggests decks thinner than 230 mm (9 in.) are more susceptible to cracking.(*9*) Missouri DOT reports that decks thicker than 254 mm (10 in.) are less susceptible to cracking, (*5*) although Wisconsin DOT suggests that the probability of cracking increases as the deck thickness increases. From the recent survey, recommended thicknesses to reduce the incidence of deck cracking are 200 mm (8 in.) and 225 mm (9 in.).

Research (*29*) showed that thickening decks to 218 mm (8.6 in.) from 162 mm (6.4 in.) reduced cracking, with nearly all the reduction occurring in cracks narrower than 0.13 mm (0.005 in.). Thicker decks did not have different crack patterns. This research concluded that the reduction in cracking obtained using thickened decks is not sufficient to warrant thickened bridge decks in California (*29*).

### Reinforcing Bar Size

*NCHRP Report 297* (*30*) recommended smaller-diameter reinforcement to reduce cracking, but it did not provide specific values. Smaller bars reduce the probability of settlement cracking (*43*), and Purvis (*9*) recommended reducing the maximum bar size to 16 mm (⅝ in.).

### Bar Type

One study found structures reinforced with black bars had less cracking than those with epoxy-coated bars (*5*). Investigators (*45*) reported that using epoxy-coated bars in concrete beams increases cracking and crack width, probably because the bond strength of concrete to epoxy-coated steel is generally less than that to uncoated steel. Johnston and Zia (*46*) conducted a series of laboratory tests to measure the bond characteristics of epoxy-coated steel reinforcing bars and found that epoxy-coated bars developed 85 percent of the bond stress of uncoated bars.

Treece and Jirsa (*45*) reported the influence of bar size, concrete strength, casting position, and epoxy thickness on the bond of epoxy-coated bars. Test results showed that the epoxy-coated steel samples developed 66 percent of the bond stress of the uncoated bars, and epoxy-coated samples had fewer but wider cracks. For 20-mm (¾-in.) [No. 6] bars, the average crack width was up to twice the width of the beams with uncoated bars.

One study (*47*) did not show clear evidence of increased cracking with the use of epoxy-coated steel, but reported an increased cracking tendency. Coating thicknesses from 0.13 to 0.30 mm (0.005 to 0.012 in.) did not influence bond development length, except on small 16-mm (⅝-in.) [No. 5] bars where thicker coatings had lower bond strengths. The bar deformation pattern significantly affects the bond. The coating affects bars with larger rib bearing areas with respect to bar cross section less than bars with smaller bearing areas.

Researchers (*48*) did flexural tests of large beam specimens reinforced with either epoxy-coated or uncoated steel. Average crack widths were 50 percent larger in specimens with epoxy-coated steel. Fewer cracks developed with epoxy-coated steel specimens, but the sum of all crack widths was 25 percent higher for beams with epoxy-coated reinforcement. However, careful consideration of the corrosion-related issues should be addressed when selecting a steel type. Cracking still occurs in structures without epoxy-coated steel, and corrosion protection is totally lost at crack locations.

### Skew

Skew does not significantly affect transverse cracking, but slightly higher stresses occur near corners. One researcher (*9*) found structures with skews greater than 30 degrees to be more susceptible to transverse cracking.

### Traffic Volume

Horn (*44*) found that structures with high traffic volumes typically had more cracks than those with lower volumes, although the trend was not clearly shown. Heavy truck traffic appeared to extend crack lengths, but did not significantly affect leakage. Several other researchers found that average daily traffic did not affect deck cracking (*8,13*). High traffic volumes may ravel cracks, making them more visible.

### Alignment of Bars

Most transverse deck cracks are aligned directly above the top reinforcing bars (*6*). There is very little information relating bar alignment to deck cracking. When the top and bottom transverse bars align, they form a weakened section within the concrete that is more susceptible to cracking. Full-depth cracking usually occurs through both top and bottom transverse bars when the bars align.

### Form Type

The effect of SIP forms on cracking appears inconsistent. Some researchers found that less transverse cracking occurred when SIP forms were used (*8*), while others found no such relationship (*31*). Corrugated SIP steel forms will cause deck shrinkage that has a more linear gradient than a uniform gradient and, as shown in this project, produce larger tensile stresses at the upper deck surface. This increases the risk and severity of transverse deck cracking. The SIP forms have an added disadvantage of hiding cracks, which may prevent ready inspection for deterioration. Investigators have observed instances where SIP steel forms have corroded because of water leakage through cracks. The analytical study revealed that SIP forms typically have a small affect on deck stresses for a given applied temperature change or shrinkage profile.

### Frequency of Traffic-Induced Vibration

Traffic-induced vibrations (*12*) and vibration frequency (*11*) do not normally cause deck cracking. Investigations of bridge deck widenings did not show any problems from traffic-induced vibrations. Many bridge decks crack before they are subjected to traffic loads. Studies during this project estimated a maximum of 0.7 MPa (100 psi) tensile stress in the decks subjected to traffic vibrations.

### Girder Size and Spacing

The analytical study showed that both girder size and spacing (the tributary deck width per girder) have a moderate or small effect on stresses that develop in the deck from shrinkage and temperature changes. Generally, larger girders spaced closer together cause larger shrinkage and thermal stresses in the deck. The deck stresses are not much different when either intermediate or large girders are used, such as a large rolled steel section or plate girder, or a concrete girder at least 1.2-m (4-ft) deep, but the stresses are noticeably different when a small girder is used, such as a small steel girder, or a concrete girder 0.6-m (2-ft) deep. Selecting a lighter girder section requires a closer spacing, and the combined effects on deck stresses typically are offsetting. The risk or severity of transverse deck cracking generally increases as span length increases because a larger girder will be required.

### Design Methods

Bridge designers in the United States use AASHTO design methods almost exclusively. Transportation agencies (*49*) have suggested that shrinkage and temperature reinforcement according to AASHTO requirements are too low and should be increased. Design changes from allowable stress to load factor design may have resulted in more flexible structures that are more susceptible to cracking (*9,30*). However, one researcher found that the flexibility of the structure was unimportant (*8*).

Current AASHTO design procedure requires that bridge design account for longitudinal movement at the supports because of temperature changes, and does not require examination of shrinkage (except with prestressed bridges) or nonuniform temperatures. The current AASHTO procedure is adequate to calculate movement that the system must accommodate at supports, but it does not address tensile stresses that occur from shrinkage and daily temperature changes. This deficiency is clear because many bridge decks develop transverse deck cracks before traffic loads are applied.

AASHTO design procedures permit tensile stresses in post-tensioned decks and do not require additional reinforcement if the stresses do not exceed a certain limit; for large tensile stresses, they require that reinforcement control (not prevent) cracking. For many design requirements, designing the post-tensioning to produce tensile stresses will result in a more efficient design, and therefore AASHTO indirectly encourages tensile stresses. Especially when additional reinforcement is not placed in tensile zones created by the post-tensioning, the risk of transverse cracking is further increased.

### Summary of Design Influences on Deck Cracking

Table 1 lists the various design factors and ranks them according to their influence on cracking. General conclusions include the following:

- Cracking is more common among steel girder structures.
- Continuous-span structures are more susceptible to cracking than simple-span structures.
- SIP deck forms sometimes increase deck cracking.
- Larger stresses develop with larger girders at narrower spacing.
- Cracking stresses are less affected by bridge geometry than by concrete material properties.
- Dead-load deflections during construction should be considered.
- Cover over reinforcement should be between 38 mm (1.5 in.) and 76 mm (3 in.).
- Girder restraint and studs cause significant deck stresses.
- Thinner decks have higher stresses, and decks should not be less than 200- to 230-mm (8- to 9-in.) thick.
- Use of epoxy-coated bars has probably increased the number and width of deck cracks.
- More reinforcing bars of smaller diameter will reduce crack widths.
- Traffic-induced vibrations do not affect deck cracking.
- Designing for tensile stresses in the deck of a post-tensioned bridge will reduce early cracking.
- Reducing deck flexibility will reduce early cracking.

## EFFECT OF CONSTRUCTION PRACTICES

Sometimes construction details significantly affect early transverse cracking. Researchers (*8*) in Pennsylvania found that bridges built by two contractors had a much higher incidence of cracking than bridges built by nine other contractors. Construction factors are also shown on Table 1 and are outlined in the following sections.

### Weather and Time of Placement

Weather during concrete placement can greatly affect the number of medium and large cracks that form early in the deck; several investigators and transportation departments considered this factor to be the most significant affecting this cracking (*29*). Wind velocity affects both plastic and drying shrinkage. More cracking was observed for concrete cast during low humidities and high evaporation rates (*30*).

Transportation agencies reported that casting at night can significantly reduce deck cracking (*6,9*), and afternoon pours are most likely to crack. A survey of pavements cured with clear membranes found that cracking occurred predominantly in pavements placed in morning hours (*10*).

Weather conditions when a concrete bridge deck is placed affect residual and other thermal stresses in a bridge deck. Concrete placement at the Portland-Columbia Bridge started in early afternoon, and maximum temperatures developed in the deck just after midnight. Early peak concrete temperatures are affected by the exothermic (heat-producing) hydration of the cement and the weather. Solar radiation and air temperatures at the Portland-Columbia Bridge were warmest during and after placement and increased the temperature of the fresh concrete. The warmer concrete temperatures further increased the rate of hydration and the corresponding temperature rise caused by hydration. For most bridges, placing the concrete around noon will maximize the temperature the concrete reaches during its initial hydration and the cooling that follows. This maximizes the thermal stresses that initially develop and the risk of early cracking.

When concrete is placed during early or mid-evening, the weather immediately following the placement will cool the concrete as it is hydrating, reduce the rate of hydration, and reduce the peak temperature it reaches. Therefore, placing concrete during early or mid-evening will reduce thermal stresses and the risk of early cracking; whereas placing concrete during late morning or early afternoon will maximize the early thermal stresses and the risk of cracking.

Heat of hydration effects can also be significant during cool weather. For most bridges, placing cooler concrete during cooler weather can reduce residual thermal stresses caused by early hydration temperatures. Evaporation increases when concrete is much warmer than the ambient air temperature when it is placed. Instantaneous temperature variations in the deck are lower, as are the residual stresses that may cause or contribute to transverse cracking.

If the concrete and steel have different thermal expansion rates, seasonal temperature changes will cause stresses in the steel-girder bridge. When concrete is poured on a day that is warmer than average, bridge temperatures will be lower most of the time after the concrete has hardened. Because concrete typically expands at a lower thermal rate than steel, a uniform temperature decrease in the steel bridge will cause beneficial compressive stresses to develop in the deck. Conversely, when concrete is placed on a very cold day, warming will cause tensile stresses to develop in the deck of a steel bridge; however, these stresses are usually smaller than early hydration thermal stresses, and for most bridges placing concrete during cooler weather is beneficial.

Cracking appears worse when concrete is cast at either low temperatures (*5,31*) or high temperatures, a factor that suggests that plastic shrinkage plays a role. Florida DOT reported more cracking in decks cast between May and August, and the least cracking in decks cast in October and November. The maximum air temperature at time of casting specified by the transportation agencies ranges from 27°C (80°F) to 35°C (95°F). The minimum air temperature at time of casting specified by the transportation agencies ranges from 2°C (35°F) to 10°C (50°F). The minimum specified concrete placement temperature varies from 7°C (45°F) to 16°C (60°F). Researchers recommended (*31*) that decks not be cast when air temperature is less than 7°C (45°F). When warm concrete is cast in cool weather, the concrete heats the air above its surface and reduces its humidity. This causes increased evaporation from the concrete surface. Using a water fog mist or an evaporation retarder film is therefore important in order to prevent evaporation and plastic shrinkage cracks during cool or hot weather.

### Curing

Ineffective curing was the most common reason suggested by the transportation agencies for excessive transverse deck cracking (*6,50*). Chemical evaporation retarder films can significantly reduce the number of small deck cracks (*29*). Moist curing, using wet burlap instead of a curing compound, can result in fewer small cracks, provided the moist cure is applied early. Delayed curing increases the number of cracks. Fogging immediately after strike-off can significantly reduce early plastic deck cracking. Fogging before placing burlap often reduces initial cracking, but final cracking is usually similar in decks cured without fogging before placing burlap.

The PCA (*51*) defines curing as the maintenance of a satisfactory moisture content and temperature in concrete during some finite period immediately following placing and finishing so that the desired properties may develop. Curing has a strong influence on concrete durability, strength, watertightness, abrasion resistance, volume stability, and resistance to freezing, thawing, and deicer salts. As an example, improperly cured concrete may have an ultimate compressive strength 60 percent less than that of a fully cured specimen.

High cement content, low water-cement ratio concretes are affected more by curing than concretes with low cement contents and high water-cement ratios. Extending the wet curing to 60 days did not significantly change when the ring specimens with 278 kg/m$^3$ (470 lb/yd$^3$) of cement and a water-cement ratio of 0.5 cracked. However, when the cement content was increased to 501 kg/m$^3$ (846 lb/yd$^3$) and the water-cement ratio was decreased to 0.35, increasing the wet curing to 60 days significantly extended time-to-cracking (cracking occurred at 21 days instead of 12 days). The time-to-cracking was calculated from when the wet curing was stopped. However, this shows that even with extended wet curing, cracking can still occur under restrained conditions in the ring test (or on a composite bridge deck).

Transportation agencies in the United States specify a variety of curing techniques and curing times; moist curing is not standardized. By comparison, precast, prestressed concrete girders are cured in a standardized accelerated heat or steam manner as specified by AASHTO. A standardized AASHTO moist-curing procedure is needed for cast-in-place decks.

### Vibration of Fresh Concrete

Research and the survey results suggest that inadequate vibration is a major cause of cracking (*9*). Effective consolidation will improve all the important properties of concrete in bridge decks (*31*). Problems of under-vibration are more widespread than those of over-vibration (*31*). Areas of under-vibration are more prone to cracking (*28*). A minimum of three vibrators are recommended for placement rates of 22 $m^3$/hr (30 $yd^3$/hr). Testing showed that vibrator frequencies, size, and time of insertion can vary without changing consolidation (*31*). Revibration with a vibrating screed can close plastic shrinkage cracks.

Concrete settles during placement, vibration, and finishing. When reinforcing bars or formwork prevents concrete from settling in localized areas, voids and cracking may occur next to these areas of restraint. To control settlement cracking, it is necessary to use a concrete cover of at least 50 mm (2 in.), small-diameter reinforcing bars, and low-slump concrete.

### Finishing

Finishing procedures can affect cracking. Delayed finishing can increase cracking. Early finishing produces smaller and fewer cracks (*29,50*). Double-floated decks developed fewer small cracks than those with standard finish (*29,50*). One study found that hand finishing often increased deck cracking (*29*). Illinois DOT recommends replacing rake tining with mechanical grooving, so that curing can commence soon after placement. Sawcut grooving (1) does not damage the surface of hardened concrete when compared to rake tining of plastic concrete and (2) provides more uniform and durable grooves.

### Pour Length and Sequence

Some research suggests that placement sequence can cause deck cracking. Other research found that pour length and sequence did not influence transverse cracking (*9,11*). Deck cracking occurs with both continuously placed and sequenced pours. Some agencies recommend sequenced pours.

### Reinforcement Ties

Research (*44*) found that decks with tightly tied reinforcement often develop more cracks initially than decks with loosely tied reinforcement, but ultimately cracking was similar.

### Construction Loads

Early construction loads may cause cracking (*29*). Cracking can occur when construction machinery or other heavy loads are placed on the deck at early ages. While construction loads can cause problems, they are rarely the primary cause of transverse cracks in new bridge decks. Nevertheless, construction loads should not be allowed until the concrete has sufficiently hardened to prevent cracking.

### Traffic-Induced Vibrations

Traffic-induced vibrations before or during hardening were found not detrimental to deck concrete (*8,12,13*). Concrete disrupted by retempering, or deflections after initial set, was a cause of deck cracking (*29,30*) resulting in large cracks (*9*). However, traffic vibrations have not generally caused these deflections.

### Revolutions in Concrete Truck

No correlation between the number of revolutions of transit mix trucks and deck cracking was found (*29*).

### Summary of Construction Influences on Deck Cracking

A list of ranked construction-related factors that affect transverse deck cracking is found in Table 1. The most critical construction influences on deck cracking are weather conditions. Adverse weather includes wind, high and low temperatures, and low-humidity conditions. The evaporation rate should be measured at the jobsite, and wind breaks, fogging and evaporation retarder films should be used during periods of high evaporation during cold or hot weather. Good curing practices should always be used, especially in hot or cold weather. Early fogging and wet curing will prevent plastic shrinkage cracking, but curing alone will not prevent drying shrinkage cracking. Wet curing should be used during hot weather to cool the concrete and reduce peak temperatures. Casting at night should also be considered during hot weather.

While many agencies permit liquid-applied curing compounds during the moist-cure period, few agencies commented on their effectiveness. Because these curing compounds allow water vapor loss, they should not be the sole curing material used but should be used at early ages in combination with other highly effective moisture-retention methods, such as using sheet materials and water for the long-term curing.

Adequate vibration is necessary to prevent settlement cracking. Vibrating concrete properly is necessary, even with high-slump, superplasticized concrete. Revibration can close settlement voids and plastic shrinkage cracks, and improve the quality and appearance of concrete.

CHAPTER 3

# INTERPRETATION, APPRAISAL, APPLICATION

## HISTORY OF AASHTO SPECIFICATIONS AND CRACKING OF DECKS

Based on experience, the survey responses, and conversations with transportation officials, there is a general perception that the incidence of cracking in bridge decks increased in the mid-1970s. This increase coincides with when AASHTO made major changes to its concrete specifications.

The widespread use of deicing chemicals on roads and bridges—begun in the early 1950s—has proliferated the corrosion of conventional reinforcing steel embedded in concrete. This corrosion was first noted in about 1960. Much progress in the AASHTO specifications over the years has resulted in the design and construction of more durable, corrosion-resistant, and economical bridge structures. Summaries of pertinent information from the 1931 to 1986 AASHTO Specifications for Class A or AE (air entrainment) concrete are presented in Tables 2 and 2(a).

The minimum 28-day compressive strength of 20.7 MPa (3000 psi) the AASHTO specifications required from 1931 to 1973 was very low, with resultant low modulus of elasticity and high creep potentials. During this period, the specifications did not limit the water-cement ratio. The 1974 AASHTO Specification significantly improved corrosion protection of uncracked concrete, when the permeability of air-entrained concrete was reduced by requiring a maximum water-cement ratio of 0.44, increasing the minimum 28-day compressive strength 50 percent to 31.0 MPa (4500 psi), and increasing the minimum cement content 8 percent from 335 kg/m$^3$ to 363 kg/m$^3$ (6.0 to 6.5 bags/yd$^3$). It also increased the minimum clear cover for decks (top-of-slab) to 50 mm (2 in.).

Before 1974, the AASHTO Specifications required a slump from 50 to 100 mm (2 to 4 in.) for vibrated air-entrained Class AE concrete. Then in 1974, the specified slump was reduced to 25 mm to 65 mm (1 to 2.5 in.) in an attempt to improve concrete quality, but jobsite problems began immediately. AASHTO members and many bridge contractors challenged this slump reduction, and the 1978 AASHTO revision eliminated slump as a specified requirement. AASHTO no longer specifies slump because HRWRA are routinely used to produce concrete with high-slump, very low water-cement ratio, and high strength. Also, it was recognized that the water-cement ratio primarily dictates chloride permeability, not slump.

Bridges built before 1974 were constructed with lower strength concretes having lower elastic moduli and higher creep. A typical 20.7 MPa (3000 psi) 28-day strength concrete could have 2- and 28-day modulus of elasticity values of 9.0 and 22.0 GPa (1.3 and 3.2 $\times$ 10$^6$ psi), respectively. This 2-day modulus represents only 40 percent of the low 28-day modulus of elasticity. These concrete properties caused less deck cracking, even with composite decks.

When the 1974 AASHTO specification changes were implemented, concrete compressive strengths and elastic moduli increased significantly, and creep decreased. This increased deck cracking. Concretes produced to the 1974 AASHTO requirements have much larger modulus of elasticity values, a factor shown in this project's analytical studies to be a major factor in tensile stress development in decks. The instrumented Portland-Columbia Bridge concrete had a 2-day modulus of elasticity of 19.3 GPa (2.8 $\times$ 10$^6$ psi), 82 percent of the 28-day modulus and twice the typical value of a 20.7 MPa (3000 psi) concrete. Since 1974, high early and later concrete compressive strengths and modulus of elasticity values have been routinely used in the United States. When higher cement contents and HRWRA admixtures and silica fume are used, 1-day moist-cured compressive strengths of 27.6 to 55.1 MPa (4000 to 8000 psi) have been achieved. These concretes would have 1-day modulus of elasticity values of 28.8 to 35.8 GPa (3.6 to 5.2 $\times$ 10$^6$ psi), values 3 to 7 times those of a nominal 20.7 MPa (3000 psi) concrete used before 1974. These very high-strength concretes also have significantly reduced creep potential, behaving as what some engineers call "brittle concretes." This perceived brittleness, in fact, relates to dramatically reduced creep potential and the observed early cracking or other unusual cracking that is not consistent with engineers' experiences with more conventional concrete.

Ultra-high early compressive strengths have been produced with heat-cured, low water-to-cement ratio concretes with silica fume and silica fume-fly ash ingredients. The 1-day compressive strengths, with heat curing, and in some cases with only moist curing, can be 48.2 to 75.8 MPa (7000 to 11,000 psi). These concretes have 1-day modulus of elas-

**TABLE 2 Summary of AASHTO specifications for bridge deck concrete CLASS A or A(AE) requirements for corrosion protection (Metric) (Note: Conversion in Table 2[a])**

| Year | Class | Min. 28-day $f_c'$ (MPa) | Max. w/c | Min. cement content (kg/m$^3$) | Air content (%) | Slump* (mm) | Reinforced concrete minimum clear cover (mm) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Top of slab | Bottom of slab | Other exposed elements (main bars) | Cast-in-place seawater elements | Precast seawater elements |
| 1931 | A | 20 | -- | 362 | -- | 50-75 | 25 | 25 | 50 | 100 | 75 |
| 1941 | A | 20 | 0.53 | 362 | -- | 50-100 | 25 | 25 | 50 | 100 | 75 |
| 1945 | A | 20 | 0.53 | 334 | -- | 50-100 | 25 | 25 | 50 | 100 | 75 |
| 1953 | AE | 20 | -- | 334 | 4-7 | 50-100 | 25 | 25 | 50 | 100 | 75 |
| 1961 | AE | 20 | -- | 334 | 4-7 | 50-100 | 25 | 25 | 50 | 100 | 75 |
| 1969 | AE | 20 | -- | 334 | 4-7 | 50-100 | 37 | 25 | 50 | 100 | 75 |
| 1973 | AE | 20 | -- | 334 | 4-7 | 50-100 | 37** | 25** | 50** | 100 | 75 |
| 1974 | AE | 31 | 0.44 | 362 | 5-7 | 25-64 | 50** | 25** | 50** | 100 | 75 |
| 1974 | A | 28 | 0.49 | 362 | 3-5 | 50-100 | 50** | 25** | 50** | 100 | 75 |
| 1978 | AE | 31 | 0.44 | 362 | 5-7 | | 50** | 25** | 50** | 100 | 75 |
| 1978 | A | 28 | 0.49 | 362 | 3-5 | | 50** | 25** | 50** | 100 | 75 |
| 1983/86 | AE | 31 | 0.44 | 362 | 5-7 | | 50*** | 25** | 50*** | 100 | 75 |
| 1983/86 | A | 28 | 0.49 | 362 | 3-5 | | 50*** | 25** | 50*** | 100 | 75 |

* For vibrated concrete, if specified.

** The clear cover should be increased when "chlorides or other corrosive substances" are present. No mention of deicing chemicals in normal reinforced concrete design section.

*** In the normal reinforced concrete section, the specification states that when in a corrosive environment or marine environment, a more impervious concrete or other means should be used. The use of additional clear cover is not suggested. In the prestressed concrete design section, the specification states that when deicers are used or where saltwater, salt spray, etc., are present, additional cover should be used but suggestions about the use of a lower permeability concrete are not provided.

**TABLE 2(a) Summary of AASHTO specifications for bridge deck concrete CLASS A or A(AE) requirements for corrosion protection**

| Year | Class | Min. 28-day $f_c'$ (psi) | Max. w/c | Min. cement content (bags/yd$^3$) | Air content (%) | Slump* (in.) | Reinforced concrete minimum clear cover (in.) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Top of slab | Bottom of slab | Other exposed elements (main bars) | Cast-in-place seawater elements | Precast seawater elements |
| 1931 | A | 3000 | -- | 6.5 | -- | 2-3 | 1 | 1 | 2 | 4 | 3 |
| 1941 | A | 3000 | 0.53 | 6.5 | -- | 2-4 | 1 | 1 | 2 | 4 | 3 |
| 1945 | A | 3000 | 0.53 | 6.0 | -- | 2-4 | 1 | 1 | 2 | 4 | 3 |
| 1953 | AE | 3000 | -- | 6.0 | 4-7 | 2-4 | 1 | 1 | 2 | 4 | 3 |
| 1961 | AE | 3000 | -- | 6.0 | 4-7 | 2-4 | 1 | 1 | 2 | 4 | 3 |
| 1969 | AE | 3000 | -- | 6.0 | 4-7 | 2-4 | 1.5 | 1 | 2 | 4 | 3 |
| 1973 | AE | 3000 | -- | 6.0 | 4-7 | 2-4 | 1.5** | 1** | 2** | 4 | 3 |
| 1974 | AE | 4500 | 0.44 | 6.5 | 5-7 | 1-2.5 | 2** | 1** | 2** | 4 | 3 |
| 1974 | A | 4000 | 0.49 | 6.5 | 3-5 | 2-4 | 2** | 1** | 2** | 4 | 3 |
| 1978 | AE | 4500 | 0.44 | 6.5 | 5-7 | -- | 2** | 1** | 2** | 4 | 3 |
| 1978 | A | 4000 | 0.49 | 6.5 | 3-5 | -- | 2* | 1** | 2** | 4 | 3 |
| 1983/86 | AE | 4500 | 0.44 | 6.5 | 5-7 | -- | 2*** | 1** | 2*** | 4 | 3 |
| 1983/86 | A | 4000 | 0.49 | 6.5 | 3-5 | -- | 2*** | 1** | 2*** | 4 | 3 |

* For vibrated concrete, if specified.

** The clear cover should be increased when "chlorides or other corrosive substances" are present. No mention of deicing chemicals in normal reinforced concrete design section.

*** In the normal reinforced concrete section, the specification states that when in a corrosive environment or marine environment, a more impervious concrete or other means should be used. The use of additional clear cover is not suggested. In the prestressed concrete design section, the specification states that when deicers are used or where saltwater, salt spray, etc., are present, additional cover should be used but suggestions about the use of a lower permeability concrete are not provided.

ticity values of 33.1 to 41.3 GPa (4.8 to 6.0 × $10^6$ psi), with 28-day values similar to the 1-day values. Decks made with these concretes will have high tensile stresses because of the high early modulus and low creep.

Experience and data show that while the chloride and water permeability of these newer, low water-cement ratio concretes are dramatically reduced when the concrete is uncracked, the dramatic increase in early and later moduli of elasticity with their attendant reduction of creep results in a dramatically greater risk of high tensile stresses and cracking in restrained bridge decks.

## CRACKING OF CONCRETE

Concrete is a brittle material when subjected to tensile stresses. The tensile strain capacity of concrete is limited to about 100 to 200 $\mu\epsilon$. This lack of ductility resulted in the development of prestressed concrete such that compressive stresses induced in the concrete by prestressing steel could reduce tensile stresses and cracking. Post-tensioning is used in cast-in-place decks but rarely with bridge decks constructed on steel girders or non-post-tensioned concrete girders.

Three factors dominate the cracking of bridge decks: (1) the degree of restraint; (2) the concrete's effective modulus of elasticity that includes creep effects; and (3) concrete volume changes from shrinkage and thermal effects. A discussion of these factors follows.

### Restraint

Unrestrained concrete can undergo large negative strains from shrinkage and temperature decreases without developing tensile stresses and cracking. The same concrete, when restrained against contraction, develops tensile stresses and often results in cracks. Therefore, restraint is a major factor in the cracking of concrete decks. The restraint can be from external or internal sources. Embedded reinforcing bars provide internal restraint, and girders provide external restraint. SIP steel forms also provide external restraint. When there is restraint, shrinkage and temperature changes cause tensile stresses.

The degree of deck restraint possible with the various types of bridges constructed in the United States was studied in this project. These studies reviewed restraint based on the following commonly assumed conditions for composite behavior.

- The concrete deck is fully restrained in the transverse direction, and the girder is unrestrained in the transverse direction.
- Temperature is constant across the width and along the length of the bridge.
- The original temperature of the bridge is uniform. If the original temperature is not uniform, the effects of individual temperature changes can be determined and superimposed on other shrinkage and temperature effects.
- The later temperature of the beam is uniform.
- The later temperature of the deck is either uniform, or nonuniform linear with the soffit temperature equal to the beam temperature.
- The reinforcing steel has negligible bending stiffness.
- Separation between the concrete deck and beam does not occur.
- The curvature of the deck must match the curvature of the girder at their interface.

Equations were developed to calculate the forces and force couples in a composite beam, and the resulting stresses. Once these forces and couples are calculated, strains and stresses in the concrete deck can be determined. The derived equations accommodate multiple levels of reinforcement, to include the effects of longitudinal bars and a SIP metal form. Two sets of equations were derived to analyze strains and stresses in a composite bridge. The first set assumes a uniform equivalent temperature change in the deck and an independent uniform equivalent temperature change in the girders, as shown in Figure 4. Actual temperatures are used to study thermal behavior, whereas equivalent temperatures are used to study shrinkage behavior. When both the deck and (concrete) girders shrink, or when both undergo a temperature change, the equivalent temperatures $T_1$ and $T_2$ are non-zero. When steel girders are used and only the deck shrinks, or if only the deck undergoes a temperature change, the equivalent temperature change $T_2$ is zero. The second set of derived equations analyze strains and stresses in a composite bridge from a linear equivalent temperature change in the deck and an independent uniform equivalent temperature change in the girders matching the temperature change at the deck soffit, as shown in Figure 5.

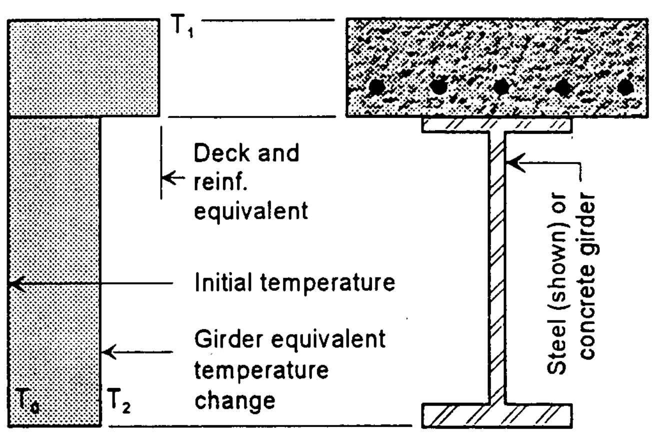


*Figure 4. Uniform temperature change in concrete deck and uniform temperature change in girder, Condition 1.*

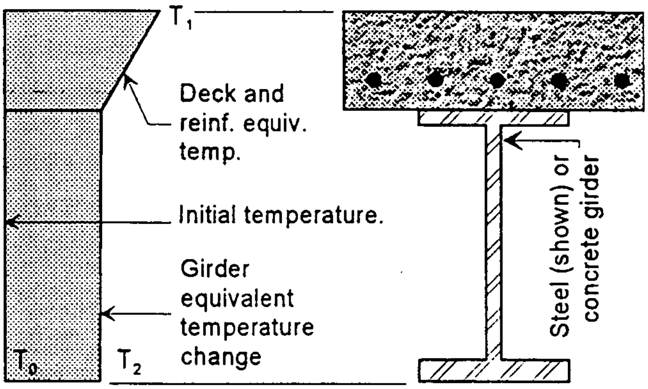


*Figure 5. Linear temperature change in concrete deck and uniform temperature change in girder, Condition 2.*

These derived equations for uniform and linear volume changes in the deck can be used to calculate the degree of restraint provided by a particular bridge design. Numerous bridge design scenarios were reviewed to determine the effect of restraint, and the following observations were made.

- Shrinkage and thermal stresses in a bridge deck depend on the restraint provided to the deck by the girders. Because girders restrain a deck at the soffit and not the centroid of the deck, the eccentric restraint causes both membrane (in-plane) and bending stresses in the deck. These effects are complicated, and stress reversals can occur when bending stresses exceed membrane stresses. Even uniform shrinkage or temperature changes can cause tensile stresses on one surface of the deck and compressive stresses on the opposite face.
- When a uniform free-strain is applied to a deck (Figure 4), the girders partially restrain longitudinal movement of the deck, and the eccentric restraint causes the deck to bend. The longitudinal restraint causes membrane stresses in the deck, and the bending causes bending stresses. For some geometries, restraint bending stresses are small and restraint stresses in the deck are nearly uniform. For other geometries, bending stresses are larger than membrane stresses, and stress reversal develops in the deck. When large steel plate girders or large concrete girders support the deck, they restrain approximately 15 to 20 percent of the uniform free-strain at the upper surface of the deck, and approximately 25 to 35 percent at the soffit. Smaller steel or concrete girders typically restrain little or no free-strain at the upper surface, and between approximately 20 and 25 percent at the soffit.
- When a linear free-strain is applied to the deck, (Figure 5), girders restrain most of the curvature that would develop if the deck were separated from the girders. Steel girders typically restrain between 85 and 95 percent of this curvature, while concrete girders usually restrain between 75 and 95 percent; larger girders restrain more curvature. For the Chapter 1 example shown in Figure 2, Condition 3, a 30-m (98.4-ft) composite span would bow 8 mm (0.32 in.), only 5 percent of the 160 mm (6.3 in.) bowing potential. However, because of an interface shear, final strains in the deck are less than the curvature restraint indicates. Large steel or concrete girders can restrain about 60 percent of the free-strain at the upper surface, and smaller girders often restrain between 35 and 45 percent at the upper surface. When linear free-strains are applied to most decks, bending stresses are larger than membrane stresses, and stress reversals develop in the deck.

These calculations show that restraint can be significant in many bridge designs, particularly with large girders. Of particular value to design engineers is the ability to select bridge systems with least restraint, based on the equations derived in this project. Most bridge decks can be assumed to have a reasonably linear strain distribution from shrinkage effects, because the top surface will usually dry quicker than the bottom surface, and for SIP metal decks the bottom deck surface has limited drying, if any.

### Concrete Effective Modulus of Elasticity

Analyses found that the concrete effective modulus of elasticity significantly affects tensile stress in the deck and cracking. Disregarding Poisson's effect, volume changes from shrinkage and thermal effects create tensile stresses directly in proportion to the concrete's effective modulus of elasticity, adjusted for creep.

While creep is usually assumed to be a long-term factor, creep of young low-strength concrete, even for short periods of time, can reduce tensile stresses in restrained concrete. Creep of high-strength concrete will also help reduce restrained tensile stresses, but not to the same extent as with low-strength concrete, because high-strength concrete has a higher instantaneous modulus and lower creep, which result in much higher effective moduli of elasticity at all ages.

Early low-strength concrete with a 1- to 2-day instantaneous modulus of elasticity of 6.9 GPa ($1 \times 10^6$ psi) could have a creep potential during an early period of 5 times the instantaneous elastic strain. Under these conditions, the effective modulus of elasticity that creates restrained tensile stresses in the concrete would be reduced to one-sixth of the elastic modulus to a value of only 1.15 GPa ($0.17 \times 10^6$ psi). For this condition, a contraction strain, if restrained, would produce only a small tensile stress because of early high creep potentials.

A similar ultra-high-strength concrete with a 1- to 2-day instantaneous modulus of 41.3 GPa ($6 \times 10^6$ psi) would have a very low creep potential of 50 percent of the instantaneous elastic strain during an early period. Under these conditions, the effective modulus of elasticity would be reduced to 67 percent of the elastic modulus to a value of 27.6 GPa ($4 \times 10^6$ psi), which is 24 times that of the low-strength concrete.

These two examples show that for an identical 1 με contraction, the low-strength concrete develops under full restraint only 1.15 kPa (0.17 psi) tensile stress whereas the ultra-high-strength concrete develops 24.8 kPa (4.0 psi) tensile stress. For a 500 με free-strain contraction, and a 50 percent restraint, these two conditions would develop 0.29 MPa (42 psi) and 6.89 MPa (1000 psi) tensile stresses, respectively.

The parameter study examined the influence of the effective concrete moduli of elasticity on shrinkage and thermal stresses, with modulus values ranging from 3.4 to 31.0 GPa (0.5 to $4.5 \times 10^6$ psi). The lowest value represents fresh concrete or low-strength concrete that has undergone substantial creep, and the larger value represents moderately high-strength concrete [42.1 MPa (6000 psi)] that has undergone no creep, or very high-strength concrete [69 MPa (10,000 psi)] that has undergone reasonable creep. More than 18,000 bridge scenarios were considered in the parameter study and maximum stresses developed for restrained bridge decks were calculated for different shrinkage and thermal conditions.

### Shrinkage Effects

Concrete shrinks when it dries. The long-term drying shrinkage values for small-sized concrete cylinders, prisms, and slabs typically range from 500 to 1000 με when stored at 50 percent relative humidity and 23°C (73°F). The authors found that the magnitude of drying shrinkage is a major factor in tensile stress development and deck cracking. The analytical studies show that for effective modulus of elasticity values of 31.0 GPa ($4.5 \times 10^6$ psi) extremely high tensile stresses greater than 2.8 to 14.0 MPa (400 to 2000 psi) can develop with as little as 100 to 500 με deck shrinkage in simple-span bridges and higher tensile stresses develop in multispan bridges. These stresses can easily crack concrete. When lower effective moduli values, representative of greater creep or lower instantaneous modulus, are used to calculate tensile stresses, lower tensile stress will occur, directly proportional to the effective moduli.

Typical shrinkage data for cylindrical concrete columns with volume-to-surface ratios of 1.0 to 6.0 are shown in Figure 6. It is quite apparent that specimen size dramatically influences the rate and magnitude of the drying shrinkage in the first 100 days. This shows that concrete bridge members with different volume-to-surface ratios will produce differential shrinkage effects during the bridge's entire life, but particularly in the first months. Therefore, even if a concrete mixture is proportioned and cured to achieve a minimum ultimate final shrinkage, differential shrinkage will occur between different-sized decks and girders.

Materials and methods to achieve minimum concrete shrinkage were reviewed because shrinkage is a major factor in tensile stress development and deck cracking. Highway agencies have made the following recommendations to achieve minimum drying shrinkage for bridge deck concrete:

- Use lower amounts of portland cement.
- Use low heat of hydration pozzolans.
- Use minimum paste volumes and minimum free-shrinkage.
- Use minimum water contents and HRWRAs.
- Use Type II and avoid Type III cement.
- Use larger-sized aggregates.
- Use good quality low-shrinkage aggregates.
- Avoid clay contaminates in aggregates.
- Avoid placement temperatures over 27°C (80°F); use ice to reduce concrete temperature.

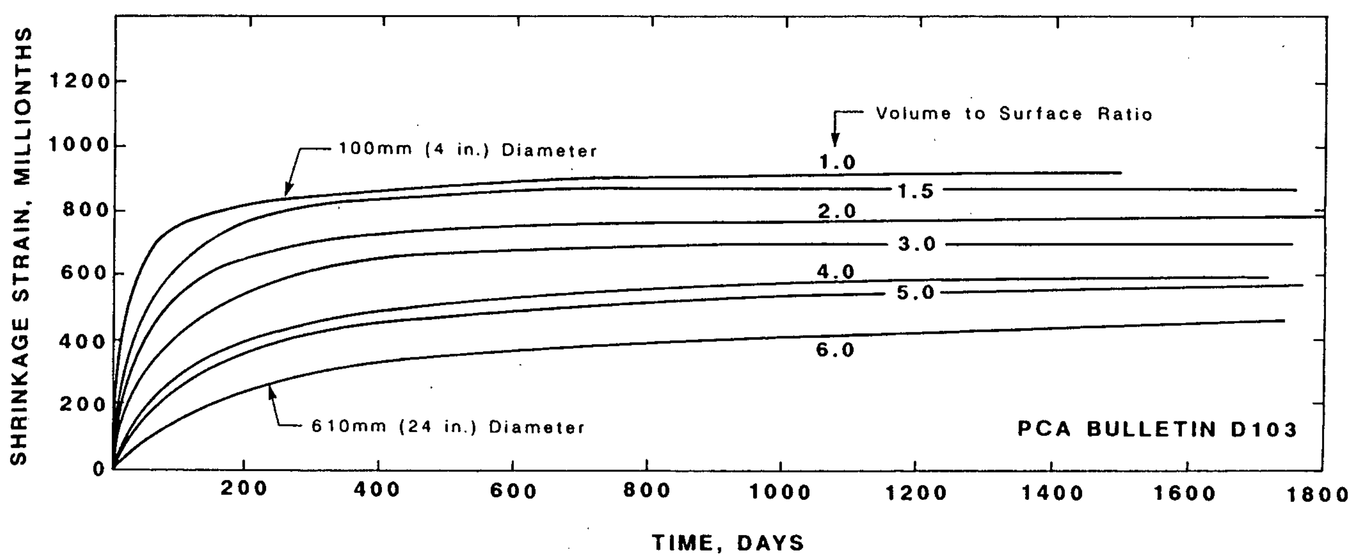


*Figure 6. Typical shrinkage data for cylindrical concrete columns with volume-to-surface ratios of 1.0 to 6.0.*

- Reduce bleeding by having a relatively smooth grading curve.
- Use shrinkage-compensating cement.
- Cast concrete at least 11°C (20°F) cooler than the ambient air temperature.
- Avoid castings in the morning and early afternoon and cast in the late afternoon or evening.
- Avoid calcium chloride and triethanolamine admixtures.
- Commence moist curing as early as possible; water fogging should be performed before initial set.

All types of cement cause concrete shrinkage, but some cause more than others. For example, a cement deficient in gypsum will shrink more than a cement with the optimum gypsum content. While concrete shrinkage can be reduced, the ultimate final shrinkage will still be a substantial value, probably at least 500 $\mu\epsilon$.

### Thermal Effects

The analytical studies indicate that tensile stresses from thermal effects are inevitable in bridge decks. Concrete and steel materials have different coefficients of thermal expansion. Steel generally has a higher coefficient of 12 $\mu\epsilon$/°C (6.7 $\mu\epsilon$/°F) when compared to concrete that has a range from 7.2 to 12.6 $\mu\epsilon$/°C (4 to 7 $\mu\epsilon$/°F), depending largely on aggregate type.

**Hydration Temperature Stresses.** The first thermal stresses develop in the deck within the first 1 or 2 days after placement. During this time, the cement is hydrating and generating substantial heat; this heat gain is larger with thicker members. Temperatures then cool to match surrounding air or structure temperatures. When concrete is still plastic or of very low strength, it can adjust to changing temperatures without developing significant stresses; however, after hardening, temperature changes cause stresses.

When a deck is cast monolithically with concrete girders, thermal stresses caused by hydration are often minimized because both the deck and girders generate hydration heat and then cool and the temperature difference is minimized. Stresses develop in this bridge type from hydration, because the deck usually cools quicker than the more massive girders.

Thermal stresses from hydration are worse in a steel-girder bridge, and worst in a concrete bridge when the deck is cast after the girders. Concrete conducts heat slower than steel, and the greater mass of the girders will cause the girders to respond slower to the changing deck temperatures. Steel girders typically conduct heat quicker than concrete girders, and upper flanges will warm and cool more with the deck, reducing the temperature difference at the interface. A 28°C (50°F) temperature change in the deck relative to the girders can cause stresses greater than 1.4 MPa (200 psi) when the young concrete has an effective modulus of elasticity of only 3.4 GPa ($0.5 \times 10^6$ psi), and greater than 6.9 MPa (1000 psi) when the effective modulus is 17.2 GPa ($2.5 \times 10^6$ psi). The Portland-Columbia Bridge concrete had a 1-day instantaneous modulus of 19.2 GPa ($2.79 \times 10^6$ psi), just as the heat of hydration cooling period finished.

**Diurnal Temperature Stresses.** For most bridges, diurnal temperature changes within the bridge produce the largest thermal stresses. The diurnal temperature cycles of the bridge decks usually exceed the ambient air temperature cycle, especially on surfaces directly exposed to solar radiation. Bridge decks in moderate or extreme climates can easily experience 28°C (50°F) diurnal temperature cycles. The upper surface of the deck typically heats and cools quicker, because it is exposed to direct solar radiation and precipitation. Because heat does not transfer instantly to the girders, temperatures are rarely uniform in a bridge, and temperature gradients usually exist. The parameter study analyses revealed that a linear temperature gradient in the deck, not a uniform temperature gradient, typically produces the largest deck stresses and the greatest risk of transverse cracking.

In a simply supported bridge, thermal tensile stresses in the deck may reach 9.3 MPa (1350 psi) with steel girders, and 10.2 MPa (1480 psi) with concrete girders. Diurnal thermal stresses are often larger over the interior supports of a continuous-span structure. Thermal tensile stresses above these interior supports may exceed 9.6 MPa (1400 psi) with steel girders and nearly 13.8 MPa (2000 psi) with concrete girders. All of these stresses are sufficient to cause transverse deck cracking, especially over the interior supports of a continuous-span structure.

**Seasonal Temperature Stresses.** Seasonal temperature changes are small or negligible in concrete bridges because both deck and girders typically have similar or the same thermal expansion rates. Deck stresses in concrete bridges caused by seasonal (uniform full-depth) temperature changes only occur because of the expansion difference of the deck reinforcing steel.

When steel girders support the concrete deck, seasonal temperatures will cause thermal stresses for those types of concrete that do not have the same thermal expansion rate as steel. Since most concretes have a lower coefficient of thermal expansion than steel, seasonal temperature decreases will generally cause compressive stresses to develop in the deck, and temperature increases will cause tensile stresses in the deck. A uniform full-depth temperature change in a steel bridge may cause stresses as large as 2.0 MPa (284 psi) in a simply supported span, and 2.0 MPa (291 psi) over interior supports of a continuous bridge.

**Factors Affecting Thermal Stresses.** Many factors affect bridge temperatures. The primary factors include the concrete material itself, geometry of the bridge, construction techniques, and environment.

Material properties—Thermal stresses that develop from seasonal (full-depth) temperature changes are linearly proportional to the different material thermal expansion rates. For temperature changes in the deck only, thermal stresses are linearly proportional to the concrete coefficient of thermal expansion. If this expansion rate is reduced 10 percent, thermal stresses are also reduced by this same amount. Generally, the combined thermal stresses increase if the concrete coefficient of thermal expansion increases.

The concrete effective modulus of elasticity affects thermal stresses in the deck; larger moduli typically cause large stresses. Poisson's ratio also affects stresses, and larger ratios typically produce larger deck stresses.

Geometry—Geometry affects thermal stresses less than material properties. Generally, larger deck stresses develop with larger girders at a narrower spacing, and with thinner decks, but many exceptions occur. Deck reinforcement has a small affect on thermal stresses in the deck. Steel studs or channels used with steel girders increase adjacent stresses approximately 20 percent.

Construction techniques—Construction can significantly affect initial temperatures and thermal stresses. The exothermic hydration of the cement and weather affect temperatures during the first 1 or 2 days after placement. When a bridge deck is placed between morning and late afternoon, warm air temperatures and high solar radiation typically heat the concrete as it is setting. As the concrete warms, the rate of hydration and the heat generated increases. Temperatures in the deck would be warmer under these conditions than if the concrete had been placed during early evening when weather cools the concrete and decreases the final temperature. Reducing the peak hydration temperature reduces initial thermal stresses and the risk of early transverse deck cracking.

With steel-girder bridges, placing the deck on a warm day (early or mid-evening preferred), typically causes compressive stresses in the deck on cooler days. These stresses will offset other shrinkage and thermal stresses, and reduce the risk of transverse deck cracking. For most steel-girder bridges, the risk of transverse deck cracking caused by seasonal temperature changes is reduced when the concrete is placed during warm summer days, and the risk increases when the concrete is placed during cold winter days.

Environment—Thermal stresses and the risk of transverse deck cracking are greatest when large seasonal temperature differences exist, solar radiation is high, and diurnal temperatures cycles are large. These conditions vary greatly by geographical location and are unavoidable. Additional care is necessary to prevent or reduce cracking in those geographical areas that produce large bridge temperature changes.

Materials and methods to achieve minimum concrete thermal volume changes were reviewed because these thermal effects are major factors in tensile stress development and deck cracking. The following recommendations are made to achieve minimum thermal effects for bridge decks:

- Use lower amounts of portland cement.
- Use low heat of hydration portland cements and pozzolans.
- Use minimum paste volumes.
- Use larger-sized aggregates.
- Use aggregates with low coefficients of expansion.
- Avoid placement temperatures over 27°C (80°F); use ice to reduce concrete temperature.
- Cast concrete at temperatures at least 11°C (20°F) cooler than the ambient air temperature.
- Avoid castings in the morning and early afternoon and use late afternoon or evening castings.
- Minimize solar radiation effects on bridge deck concrete during casting.
- Specify bridge deck concrete based on 56- or 90-day compressive strengths to allow lower heat of hydration cementitious concrete systems with pozzolans to be used.

Although concrete thermal effects can be reduced, the coefficient of thermal expansion will still be substantial because its value is determined dominantly by the aggregate type, a factor that cannot generally be controlled. More control can be exercised over the early heat of hydration effects.

### Stresses from Shrinkage and Thermal Effects

Two systems of equations were derived to analyze strains and stresses in a composite bridge. The first system assumes a uniform equivalent temperature change in the deck and an independent uniform equivalent temperature change in the girders, as shown in Figure 4.

Shrinkage and temperature changes can produce identical stresses in a bridge. Shrinkage stresses can be calculated with the derived equations by using equivalent temperature changes instead of actual temperature changes. These equivalent temperature changes, when multiplied by the (arbitrary) material coefficient of thermal expansion, must produce the same free-strain (the strain that would develop if the deck and girders were perfectly separated) as the shrinkage.

The second system analyzes strains and stresses in a composite bridge from a linear equivalent temperature change in the deck, and an independent uniform equivalent temperature change in the girders matching the temperature change at the deck soffit, as shown in Figure 5.

The following is a summary of the maximum deck stresses calculated for simply supported steel-girder bridges for the various thermal and shrinkage studies. A temperature change of $\pm 28°C$ ($\pm 50°F$) and a free-shrinkage strain of 100 $\mu\epsilon$ (the maximum stresses from a 500 $\mu\epsilon$ free-shrinkage are also provided) were considered in this analysis.

| Temperature or shrinkage, simply supported steel girder bridge | Maximum Deck Stresses | |
|---|---|---|
| | Tension | Compression |
| 28°C (50°F) uniform **increase**, full section | 1.9 MPa (284 psi) | 1.5 MPa (221 psi) |
| 28°C (50°F) uniform **decrease**, full section | 1.5 MPa (221 psi) | 1.9 MPa (284 psi) |
| 28°C (50°F) uniform **increase**, deck section only | 2.2 MPa (313 psi) | 5.9 MPa (861 psi) |
| 28°C (50°F) uniform **decrease**, deck section only | 5.9 MPa (861 psi) | 2.2 MPa (313 psi) |
| 28°C (50°F) linear **increase**, deck section only | 5.9 MPa (851 psi) | 9.3 MPa (1352 psi) |
| 28°C (50°F) linear **decrease**, deck section only | 9.3 MPa (1352 psi) | 5.9 MPa (851 psi) |
| 100 $\mu\epsilon$ uniform shrinkage | 1.9 MPa (279 psi) | 0.6 MPa (86 psi) |
| 100 $\mu\epsilon$ linear shrinkage | 2.8 MPa (407 psi) | 1.6 MPa (238 psi) |
| 500 $\mu\epsilon$ uniform shrinkage | 9.6 MPa (1395 psi) | 3.0 MPa (430 psi) |
| 500 $\mu\epsilon$ linear shrinkage | 14.0 MPa (2035 psi) | 8.2 MPa (1190 psi) |

The following is a summary of the maximum stresses calculated in the deck of a simply supported, cast-in-place concrete-girder bridge for the thermal and shrinkage studies, with an applied temperature change of 28°C (50°F) and applied free-shrinkage differential strain of 100 $\mu\epsilon$ between the deck and the concrete girders. (The maximum stresses from a 500 $\mu\epsilon$ free-shrinkage differential are also provided.)

| Temperature or shrinkage, simply supported concrete girder bridge | Maximum Deck Stress | |
|---|---|---|
| | Tension | Compression |
| 28°C (50°F) uniform **increase**, full section | 0.69 MPa (100 psi) | 1.38 MPa (200 psi) |
| 28°C (50°F) uniform **decrease**, full section | 1.38 MPa (200 psi) | 0.69 MPa (100 psi) |
| 28°C (50°F) uniform **increase**, deck section only | 2.59 MPa (375 psi) | 7.89 MPa (1144 psi) |
| 28°C (50°F) uniform **decrease**, deck section only | 7.89 MPa (1144 psi) | 2.59 MPa (375 psi) |
| 28°C (50°F) linear **increase**, deck section only | 5.70 MPa (827 psi) | 10.2 MPa (1480 psi) |
| 28°C (50°F) linear **decrease**, deck section only | 10.2 MPa (1480 psi) | 5.70 MPa (827 psi) |
| 100 $\mu\epsilon$ uniform deck shrinkage | 2.42 MPa (351 psi) | 0.72 MPa (104 psi) |
| 100 $\mu\epsilon$ linear deck shrinkage | 3.0 MPa (441 psi) | 1.5 MPa (219 psi) |
| 500 $\mu\epsilon$ uniform deck shrinkage | 12.1 MPa (1755 psi) | 3.59 MPa (520 psi) |
| 500 $\mu\epsilon$ linear deck shrinkage | 15.2 MPa (2205 psi) | 7.55 MPa (1095 psi) |

As shown, shrinkage and temperature changes can cause a wide range of stresses in all types of simply supported bridges. Usually, a linear free-strain distribution in the deck (from temperature or shrinkage) produced the largest stresses, and the full-depth uniform free-strains produced the smallest stresses. For most geometry and material combinations, a temperature decrease will cause tensile stresses in the deck, as will shrinkage of the deck. Usually shrinkage and temperature stresses in a concrete bridge are similar to those in a steel-girder bridge, although maximum values are slightly higher in the concrete bridge.

The following is a summary of the maximum total stresses that would develop in the concrete deck of the continuous steel-girder bridge for the thermal and shrinkage studies for a temperature change of 28°C (50°F) and applied free-shrinkage strain of 100 $\mu\epsilon$. (The maximum stresses from a 500 $\mu\epsilon$ shrinkage are also provided.)

| Temperature or shrinkage, multispan steel girder bridges | Maximum Total Stress | |
|---|---|---|
| | Tension | Compression |
| 28°C (50°F) uniform **increase**, full section | 2.0 MPa (291 psi) | 1.55 MPa (225 psi) |
| 28°C (50°F) uniform **decrease**, full section | 1.55 MPa (225 in.) | 2.0 MPa (291 in.) |
| 28°C (50°F) uniform **increase**, deck section only | 1.38 MPa (200 psi) | 6.08 MPa (882 psi) |
| 28°C (50°F) uniform **decrease**, deck section only | 6.08 MPa (882 in.) | 1.38 MPa (200 in.) |
| 28°C (50°F) linear **increase**, deck section only | 0.10 MPa (14 psi) | 9.73 MPa (1412 psi) |
| 28°C (50°F) linear **decrease**, deck section only | 9.73 MPa (1412 in.) | 0.10 MPa (14 in.) |
| 100 $\mu\epsilon$ uniform shrinkage | 1.96 MPa (284 psi) | 0.39 MPa (57 psi) |
| 100 $\mu\epsilon$ linear shrinkage | 2.91 MPA (422 psi) | 0.13 MPa (19 psi) |
| 500 $\mu\epsilon$ uniform shrinkage | 9.78 MPa (1420 psi) | 1.96 MPa (285 psi) |
| 500 $\mu\epsilon$ linear shrinkage | 14.54 MPa (2110 psi) | 0.59 MPa (95 psi) |

Except for uniform full-depth temperature changes, both temperature decreases and shrinkages cause tensile continuity stresses over the interior supports. Because the corresponding simple stresses are either tensile or compressive, total stresses are larger than simple stresses with some parameter combinations, and are less with other combinations. As such, some steel-girder bridges will crack more near interior supports, and others will crack less. For the most part, the same parameters that have a large effect on simple stresses also have a large effect on total stresses.

The following is a summary of the maximum total stresses that would develop in the concrete deck of a multispan, cast-in-place, reinforced concrete bridge over the interior supports for the thermal and shrinkage studies, as described in the preceding sections.

| Temperature or shrinkage, multispan concrete girder bridges | Maximum Total Stress | |
|---|---|---|
| | Tension | Compression |
| 28°C (50°F) uniform **increase**, full section | 0.27 MPa (39 psi) | 2.25 MPa (327 psi) |
| 28°C (50°F) uniform **decrease**, full section | 2.25 MPa (327 psi) | 0.27 MPa (39 psi) |
| 28°C (50°F) uniform **increase**, deck section only | none | 13.5 MPa (1958 psi) |
| 28°C (50°F) uniform **decrease**, deck section only | 13.5 MPa (1958 psi) | none |
| 28°C (50°F) linear **increase**, deck section only | none | 13.6 MPa (1969 psi) |
| 28°C (50°F) linear **decrease**, deck section only | 13.6 MPa (1969 psi) | none |
| 100 $\mu\epsilon$ uniform shrinkage | 3.86 MPa (560 psi) | none |
| 100 $\mu\epsilon$ linear shrinkage | 3.88 MPa (563 psi) | none |
| 500 $\mu\epsilon$ uniform shrinkage | 19.29 MPa (2800 psi) | none |
| 500 $\mu\epsilon$ linear shrinkage | 19.40 MPa (2815 psi) | none |

A wide variety of very high total stresses can develop in the deck of a continuous steel or cast-in-place concrete bridge over the interior supports from shrinkage and temperature changes. Almost always, a linear free-strain in the deck (from temperature or shrinkage) produced the largest stresses, and the full-depth uniform free-strains produced the smallest stresses. For most geometry and material combinations, a uniform or linear temperature decrease in the deck will cause tensile stresses in the deck over the interior supports, as will shrinkage differentials of the deck.

Curvatures develop when a bridge deck shrinks or changes temperature. For a given shrinkage or temperature change, bridges with concrete girders typically develop larger curvatures than bridges with steel girders. In continuous-span bridges, interior supports restrain this curvature and stresses develop as a result. Because bridges with concrete girders tend to develop larger curvatures, they are often affected more by continuity over supports and have larger total stresses.

The centroidal bending axis of the composite bridge section lies within the girder and not the deck. The external restraint provided by the interior supports causes bending that produces stresses in the deck that are either tensile or compressive, not both. These bending stresses are often larger than the simple-span stresses to which they are added. For continuous concrete girders, the combined total stresses in the deck over the interior support were either tensile or compressive over the full-depth of the deck, but stress reversals often occur within the deck away from support where continuity stresses dissipate. A temperature decrease in the deck always caused full-depth tensile stresses in the deck at the interior support, larger than the simple-span tensile stresses away from the support. Therefore, continuity over supports can increase transverse cracking in bridges with concrete girders.

The initial concern for most bridges, especially over interior supports, is temperature. Most bridges may undergo a 28°C (50°F) temperature increase from hydration during the first 24 to 48 hrs, followed by a temperature decrease. If the deck is cast when the concrete girders are cast, the deck will reach even higher temperatures. A temperature change of 28°C (50°F) can cause total stresses as large as 1.0 MPa (150 psi) in concrete-girder bridges, and nearly 1.38 MPa (200 psi) in continuous steel-girder bridges when the concrete effective modulus of elasticity is a mere 3.4 GPa ($0.5 \times 10^6$ psi), and greater than 5.5 MPa (800 psi) when the effective modulus is 17.2 GPa ($2.5 \times 10^6$ psi). Such stresses may cause transverse cracking.

Even if the deck does not crack during the first several days when temperatures are unstable, substantial stresses near the tensile strength of the concrete may have developed. Then, combined with additional temperature changes and shrinkage, total stresses may increase and exceed the strength of the concrete.

## EARLY DRYING AND PLASTIC CRACKING

Plastic shrinkage cracks occur while the concrete is relatively fresh and has not started to harden. They usually appear on exposed horizontal surfaces, and can occur any time that the ambient conditions (temperature, humidity, wind velocity) are conducive to rapid evaporation. Plastic shrinkage cracking generally occurs when the rate of evaporation exceeds the rate of concrete bleeding. The width of the crack at the location of the start of separation may be as much as 6.3 mm (¼ in.); however, the cracks are usually no more than 0.6 or 0.9 m (2 or 3 ft) long, and are rarely more than 50 to 75 mm (2 to 3 in.) deep. Such cracks are seldom significant structurally.

This type of cracking has become a significant problem since 1974 because of the wide usage of low water-cement ratios, latex modifiers, superplasticizers, and silica fume. Latex and HRWRAs greatly reduce the water content and therefore the bleeding capacity of concretes. The rate of evaporation can therefore more easily exceed the rate of bleeding. Silica fume intensifies the problem because HRWRAs must be used to compensate for the extreme fineness of the silica fume material. The HRWRA reduces the amount of bleed water available while the high fineness of the silica fume reduces the rate at which water moves through the concrete.

The solutions to the problem are to reduce the evaporation rate, increase the bleeding capacity of the concrete, or both. Sunscreens, windbreaks, fog mist, or monomolecular evaporation retarder films can reduce evaporation to various degrees. The most effective means of reducing evaporation is with impermeable curing covers such as polyethylene sheeting. However, this method can be cumbersome for decks subjected to wind and adjacent traffic. The most cost-effective method to date has been the use of a fog mist applied to the concrete surface. This must be done with a commercial grade fog nozzle, which produces a very fine mist that does not damage the concrete surface and provides coverage. Care must be exercised so that none of this fogged water becomes part of the concrete during finishing. Wetting down the forms before the concrete is placed is also helpful. Increasing the bleeding capacity of the concrete is usually not practical; however, water-reducing admixtures containing hydroxylated carboxylic acid often increase bleeding.

It is well-known that on hot days evaporation occurs, but it is not generally recognized that more severe evaporation may occur during cooler weather. High evaporative conditions can occur in cold weather as well as in warm weather. Concrete also takes longer to harden in cool weather so it is subjected to evaporation in the plastic state for a longer time than when cast in warm weather. Plastic shrinkage cracks can be more severe if warm concrete is cast in winter conditions. The warm concrete heats the air immediately above the surface, reducing its relative humidity. This warm moist air is replaced by cold dry air that quickly warms up and absorbs more moisture, aggravating the drying.

Figure 7 shows a nomograph by Lerch (*52*) relating air temperature, relative humidity, concrete temperature, and wind speed to evaporation. This nomograph is also found in the *ACI Manual of Concrete Practice*, Sections ACI-305 "Hot Weather Concreting" (*53*) and ACI-308 "Curing." (*54*) It is generally accepted that plastic shrinkage cracking is less likely to occur when the evaporation rate does not exceed 1 kg/m$^2$/hr (0.2 lb/ft$^2$/hr). However, this value was established for ordinary concrete mixtures and is undoubtedly not applicable to concretes with low water-cement ratio and high cement contents, or those containing additives such as superplasticizers or silica fume, which bleed less or not at all. It may be appropriate to specify a lower allowable evaporation rate for these bridge deck type concretes.

Table 3 shows relative humidities at which the evaporation rate for conventional concretes does not exceed 0.1 kg/m$^2$/hr (0.2 lb/ft$^2$/hr) for various air temperatures, concrete temperatures, and wind speeds. For example, when concrete temperatures are between 27 and 32°C (80 and 90°F) and wind

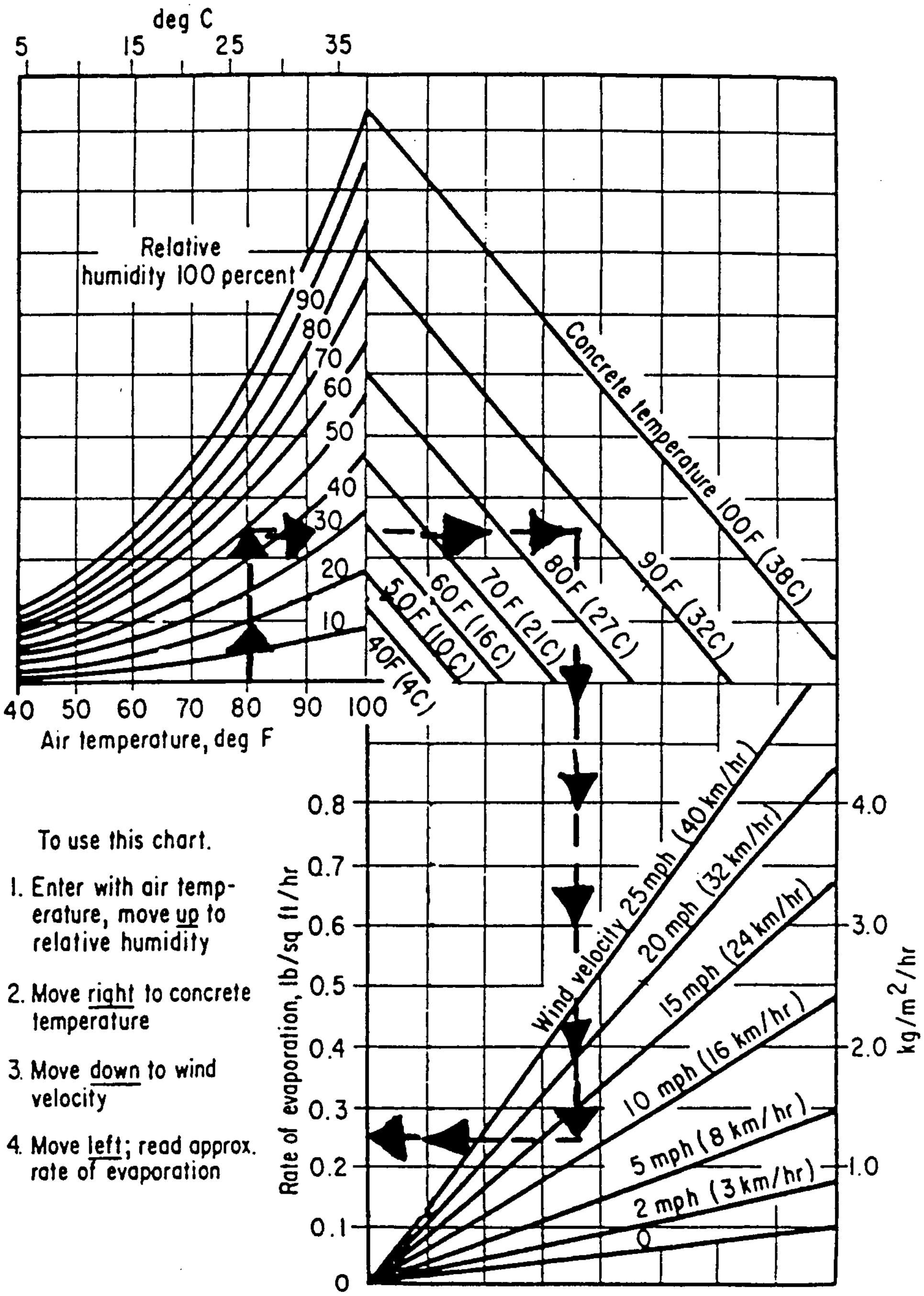


*Figure 7. Nomograph relating air temperature, relative humidity, concrete temperature, and wind speed to evaporation rate. (Lerch [49])*

speeds are a moderate 16 to 24 km/h (10 and 15 mi/h), concrete should not be placed at ambient air temperatures less than 4°C (40°F). If the concrete temperature is 32°C (90°F) it should not be placed at ambient air temperatures of 16°C (60°F) or lower, unless immediate fog curing can maintain the concrete in a saturated condition.

The nomograph does not describe wind speeds in simple terms. Table 4 shows the Beaufort scale for wind speeds and a general description of the wind. A gentle breeze is typically 16 km/h (10 mi/h) and a moderate breeze is 24 km/h (15 mi/h). With wind speeds above 16 km/h (10 mi/h), a gentle breeze, strict control of temperature, humidity, and placement temperatures are required to reduce evaporation rates below 1 $kg/m^2/hr$ (0.2 $lb/ft^2/hr$).

If the wind speed over the concrete can be reduced to less than 8 km/h (5 mi/h) by using wind breaks, there is a low probability of plastic shrinkage cracking. Specifications should require wind breaks and immediate water fogging if the evaporation rate exceeds 1 $kg/m^2/hr$ (0.2 $lb/ft^2/hr$) for normal concretes or 0.5 $kg/m^2/hr$ (0.1 $lb/ft^2/hr$) for concretes sus-

TABLE 3 Safe placement humidity: evaporation rate less than 0.1 $kg/m^2/hr$ (0.2 $lb/ft^2/hr$)

| Air temperature, °C (°F) | Concrete temperature, °C (°F) | Wind speed, km/h, mi/h | Safe placement humidity, %* |
|---|---|---|---|
| 4 (40) | 27 (80) | 16 (10) | None |
| 4 (40) | 27 (80) | 24 (15) | None |
| 16 (60) | 27 (80) | 16 (10) | 50 - 100 |
| 16 (60) | 27 (80) | 24 (15) | None |
| 27 (80) | 27 (80) | 16 (10) | 25 - 100 |
| 27 (80) | 27 (80) | 24 (15) | 50 - 100 |
| 38 (100) | 27 (80) | 16 (10) | 15 - 100 |
| 38 (100) | 27 (80) | 24 (15) | 35 - 100 |
| 4 (40) | 32 (90) | 16 (10) | None |
| 4 (40) | 32 (90) | 24 (15) | None |
| 16 (60) | 32 (90) | 16 (10) | None |
| 16 (60) | 32 (90) | 24 (15) | None |
| 27 (80) | 32 (90) | 16 (10) | 60 - 100 |
| 27 (80) | 32 (90) | 24 (15) | 88 - 100 |
| 38 (100) | 32 (90) | 16 (10) | 37 - 100 |
| 38 (100) | 32 (90) | 24 (15) | 47 - 100 |

* Values less than these levels will increase the probability of deck cracking due to excessive evaporation.

**Note:** To prevent cracking from occurring at the various air temperatures, placement temperatures, and wind speeds, the RH must not exceed values given.

ceptible to plastic cracking such as those containing low water-cement ratios, silica fume, or HRWRA.

## CONCRETE DRYING SHRINKAGE

The loss of mixing water from newly cast concrete during exposure to air at less than 100 percent relative humidity (RH) causes drying shrinkage. Starting from the saturated condition, the relation between water lost and unrestrained drying shrinkage of new concrete is roughly linear through two distinct phases. In the first phase, the water lost consists primarily of free water, and shrinkage is small. During the second phase, absorbed water is lost and shrinkage can be large. The absorbed water is contained in capillary and gel pores. Drying shrinkage of unreinforced, unrestrained newly cast concrete in a 23°C (73°F), 50 percent RH environment can range from about 500 to 1000 $\mu\epsilon$. Concrete shrinkage in air having higher or lower RH can be much lower or higher than the typical range stated above.

TABLE 4 Beaufort scale of wind speeds

| Beaufort Scale | km/h (mi/h) | Description |
|---|---|---|
| 0 | 0 | Calm |
| 1 | 3 (2) | Light |
| 2 | 8 (5) | Light breeze |
| 3 | 16 (10) | Gentle breeze |
| 4 | 24 (15) | Moderate breeze |
| 5 | 32 (20) | Fresh breeze |
| 6 | 40-50 (25-31) | Strong breeze |
| 7 | 51-61 (32-38) | Moderate gale |
| 8 | 63-74 (39-46) | Fresh gale |
| 9 | 75-87 (47-54) | Strong gale |
| 10 | 88-101 (55-63) | Whole gale |
| 11 | 103-117 (64-73) | Storm |
| 12 | >120 (>74) | Hurricane |

Most of the drying shrinkage of the deck will occur within the first year after construction. Because decks with SIP metal forms only dry from one surface, they will take longer to dry than decks with removable forms.

The large surface-to-volume ratio makes bridge decks especially susceptible to rapid drying. Within the concrete, the rate of moisture loss varies inversely as the square of the distance from the nearest drying surface. The higher the surface-to-volume ratio, the faster the concrete members will dry and shrink. With rapid drying rates, a large degree of strain differential can develop in modern, low permeability because of the very slow diffusion of mix water toward surfaces. This is a result of the surface drying and shrinking, while the interior portions of the member remain at a high moisture content and therefore shrink much less. This can produce compressive stresses in the interior and tensile stress near the surface.

ASTM C596 (*55*) can be used to determine the effect of portland cement on the drying shrinkage of a graded Ottawa sand mortar subjected to standard temperature, RH, and evaporation rates. However, the drying shrinkage of paste or mortar may not be good predictors of the drying shrinkage of concrete.

Careful selection of concrete materials can reduce drying shrinkage. Aggregate type and grading, water content, cement content, concrete placement temperature, and curing all affect performance. To reduce the overall shrinkage, designers should specify the largest practical maximum aggregate size should be used, the aggregates and cement should have low shrinkage characteristics, fines and clay material passing the 200 mesh should be a minimum, and the least amount of cement or cementitious materials to achieve the required compressive strength should be used.

## CURING

Ineffective moist curing was the most common reason suggested by the transportation agencies for excessive transverse deck cracking (*6,50*). The first several days are critical to the strength and durability of the deck. The survey revealed that curing practices varied widely and that many DOTs do not have a standard method of curing bridge decks. Many agencies allow only membrane or curing compounds, while others require curing compounds and long-term wet curing. Some specifications allow the contractor to select curing options; an option that may encourage selecting the least expensive, but not necessarily an effective curing method.

Curing has a strong influence on the properties of hardened concrete such as durability, strength, watertightness, abrasion resistance, volume stability, and resistance to freezing and thawing and deicer salts.

Evaporation retarders can significantly reduce the number of small deck cracks that form (*28*). Very early plastic deck cracking can be significantly reduced if fogging or evaporation retarder films are used immediately after strike-off. Moist curing, using wet burlap instead of a curing compound, can result in fewer small cracks, provided the moist cure is applied early. Conversely, delayed water curing increases the number of cracks.

Curing appears most important to high cement content, low water-cement ratio concretes. Extended wet curing of 60 days did not significantly change the time-to-cracking of ring specimens having 278 kg/m$^3$ (470 lb/yd$^3$), and a water-cement ratio of 0.5. However, the average time-to-cracking of the mix with 501 kg/m$^3$ (846 lb/yd$^3$) cement and 0.35 water-cement ratio was extended from 11.7 days to 21.0 days, with 60 days of wet curing.

### Continuous Moist Curing

Continuous moist curing is performed by applying a continuous water mist, by water ponding, or by applying saturated coverings such as wet burlap. With the high cement contents and the low water contents in modern concrete mixes, wet curing is the best method to reduce the evaporation of the already limited amount of mix water. Wet curing also cools the concrete, reducing the thermal stresses that result from the heat of hydration. Absorbent material should be prewetted before placing on the concrete to prevent the wicking of moisture from the concrete. This can be difficult to do in the field because of the fabric's excessive weight when wet. Therefore, prewetting the deck and immediate wetting of the installed dry fabric is sometimes done. Once started, care should be taken to prevent drying. Special attention to maintain the covering on the sides and edges of the deck must be given to prevent the wind from uncovering the edges and allowing the concrete to dry.

### Membrane Curing

Membrane curing consists of spraying a chemical compound on the surface of the concrete to prevent drying. It has the advantage of being applied sooner than wet curing blankets and its effectiveness does not end abruptly. Curing compounds should conform to the requirements of ASTM C309. White-pigmented curing compounds should be used in hot weather to reduce the temperature of the concrete.

Application of white-pigmented curing compound as soon as possible after finishing followed by continuous wet curing is preferred. The surface of the concrete should still be damp when the curing compound is applied. In combination, the curing compound will reduce the initial and later drying, and wet curing will prevent moisture loss and cool the concrete.

Sheet curing by placing plastic sheeting over the concrete is only effective if the concrete is kept continuously wet under the plastic. Plastic sheeting can be used to help keep wetted burlap or fabric moist, but plastic should not be the only method for curing decks.

### Standardization of Curing

The environment during casting must be controlled to produce more consistent and uniform bridge decks. AASHTO has standard curing procedures for precast, prestressed concrete bridge elements but not for cast-in-place bridge decks. Unfortunately, controlling the environment around the bridge when the deck is cast is often difficult. Building an enclosure around the bridge during casting would allow control of the environment. Enclosures would greatly reduce the risk of plastic shrinkage cracking by reducing solar radiation, wind speed, and evaporation rate. Controlling the moisture and temperature within the enclosure could achieve optimum curing.

Optimum curing should include the following:

- Use of fog nozzle water spray in hot weather to cool concrete and to cool the steel and forms immediately ahead of the placement—ponding of water on the forms or plastic concrete should not be allowed;
- Use of windbreaks and enclosures when the evaporation rate exceeds 1 kg/m$^2$/hr (0.2 lb/ft$^2$/hr) for normal con-

cretes or 0.5 kg/m$^2$/hr (0.1 lb/ft$^2$/hr) for low water-cement ratio concretes susceptible to plastic cracking;
- Application of water mist or monomolecular film immediately after strike-off or early finishing;
- Application of white-pigmented curing compound as soon as bleed water diminishes;
- Application of prewetted burlap as soon as the concrete resists indentation—the burlap must be kept continuously wet by continuous sprinkling or by covering the burlap with plastic sheeting and periodic sprinkling; and
- Continuation of wet curing for a minimum of 7 days, preferably 14 days—curing should be extended in cold weather until the concrete has gained adequate strength.

Diamond sawcut grooving of the hardened concrete is the preferred method of obtaining skid resistance since it will allow the application of burlap sooner than a tined surface and provides a more durable surface. Improved curing enhances strength and durability of concrete. Good curing and casting procedures should reduce plastic and early thermal cracking but will not eliminate transverse deck cracking.

## RECOMMENDATIONS TO REDUCE EARLY TRANSVERSE CRACKING

### Concrete Mix Design

DOTs should identify and specify concrete mixtures with low cracking tendencies for deck concretes. They should perform cracking-tendency ring tests on the proposed concrete mix designs to identify mixes least likely to crack.

The AASHTO-specified maximum aggregate size, water-cement ratio, and minimum cement contents for Class A and Class A (AE) concretes are in part responsible for the deck cracking that increased after the mid-1970s. The maximum aggregate size should not be limited; instead, the largest possible aggregate size should be used. The maximum aggregate size should be the smaller of one-third the deck thickness, or three-fourths the minimum clear spacing between bars. Most bridge decks can and should be constructed with aggregate of at least 40 mm (1½ in.) in size; even larger aggregates should be used when possible. Larger aggregates permit a leaner mix (lower cement paste content) while maintaining workability. Leaner mixes typically shrink less and are thermally less expansive, reducing shrinkage and thermal stresses. They also produce less heat during hydration, and develop lower thermal stresses. Leaner mixes can have low permeability, be resistant to freeze-thaw damage, and be more economical to produce.

The AASHTO specifications should not stipulate a minimum cement content, and contractors should be allowed to minimize the amount of cement or cementitious materials, as long as the concrete still meets strength and durability requirements. Type III (high early strength) cement may increase deck cracking. Decks constructed with concrete made with this cement will reach warmer hydration temperatures and develop larger thermal stresses than conventional concretes.

Concretes that gain strength slowly should be specified and used. Concrete mixes with low effective modulus of elasticity and highest early creep should be selected because they will have the lowest stress for a given strain. Low 7-day strength and moderate 60- or 90-day strengths should be specified to prevent concrete from developing a high effective modulus from the early shrinkage and thermal strains. Fly ash is beneficial because it reduces early strength without sacrificing later strength and durability. The effect of accelerating and retarding admixtures is uncertain, but HRWRAs probably should be encouraged. Silica fume admixtures appear to increase cracking.

### Longitudinal Deck Reinforcement

Additional longitudinal reinforcement can control transverse deck cracking. Additional reinforcement will reduce crack widths and improve serviceability. As a minimum, it is recommended that longitudinal size 10M (slightly smaller than a #4 bar) bars be placed at a maximum spacing of 150 mm (6 in.). This recommendation more than doubles the area of reinforcement currently required by AASHTO, 265 mm$^2$/m (0.125 in$^2$/ft), and reduces the maximum spacing by approximately one-third for most decks. Increasing the longitudinal reinforcement over supports of continuous-span bridges may not be practical, because reinforcement at these areas is often heavily congested.

### Post-Tensioned Bridge Design

AASHTO design currently allows tensile stresses in the longitudinally post-tensioned concrete deck of a bridge; doing so usually produces a design with high flexural efficiency. Any such tensile stresses, when combined with inevitable shrinkage and thermal stresses, may cause additional or more severe transverse deck cracking. For this reason, when it is feasible, designers should not design the post-tensioning to cause tensile stresses in the deck. When it is not feasible to do so, designers should consider the effects of the tensile stresses on serviceability, especially when the bridge is in a corrosive environment.

### Time of Placement

Concrete bridge decks should be placed during early or mid-evening whenever possible. Doing so will reduce hydration temperatures and the resulting thermal stresses, early shrinkage, and the risk or severity of transverse bridge deck cracking.

### Concrete and Air Temperatures at Time of Placement

Reducing concrete placement temperatures decreases hydration rates and lowers the early hydration temperature cycle and resulting thermal stresses. Corresponding thermal stresses are lower, as is the risk of transverse cracking.

This risk is minimized when concrete temperatures are as low as possible. From the agency survey, the minimum specified concrete temperature at placement ranged from 7°C (45°F) to 16°C (60°F). Limits on the maximum concrete temperature are generally 32°C (90°F), but occasionally lower.

The maximum air temperature at time of casting specified by the transportation agencies ranged from 27°C (80°F) to 35°C (95°F). The minimum air temperature at time of casting specified by the transportation agencies ranged from 2°C (35°F) to 10°C (50°F).

It is recommended that concrete be cast 5°C (10°F) to 10°C (20°F) cooler than ambient, except when temperatures are below 16°C (60°F), where the concrete should be cast at ambient. To reduce concrete temperatures, aggregates should be shaded before mixing, and ice may be used as a portion of the mix water.

### Placement Weather and Curing

Concrete should not be placed on windy days, especially when air temperatures are hot or very cold. These conditions accelerate concrete drying, and surface drying may occur before curing can be started.

It is essential that proper curing techniques be employed as soon as possible to minimize surface drying. Standardized curing techniques are needed for bridge decks. Erect windbreaks and immediately mist the concrete with water when the evaporation rate exceeds 1 kg/m$^2$/hr (0.2 lb/ft$^2$/hr) for normal concretes or 0.5 kg/m$^2$/hr (0.1 lb/ft$^2$/hr) for low water-cement ratio concretes susceptible to early-age cracking. Curing should include misting, curing compound, and wet blanket curing procedures. Wet curing should be started as soon as possible, and maintained for at least 7 days, preferably 14 days.

### Finishing and Tining

Finishing procedures affect cracking. Delaying finishing often increases cracking. Mechanical grooving of the hardened concrete damages the surface of the concrete less than rake tining during finishing and provides more uniform and durable grooves. Mechanical grooving of the hardened concrete also allows the application of the curing compound and the wetted burlap much sooner because there is no concern for damaging the tined surface.

### Structure Type

Restraint is a major factor influencing the amount of deck cracking. Cast-in-place concrete girders shrink and restrain deck shrinkage less than steel girders do. With precast concrete girders, placing the deck when the girders are still young helps compatibility since the deck shrinkage occurs during shortening of the girders because of shrinkage and creep. Deep steel-girder bridges are the most susceptible to deck cracking. Simple-span designs should be chosen when possible to reduce the deck stresses.

## CRACK REPAIR

As discussed, there is no consensus regarding the sizes of acceptable and unacceptable cracks. Typical acceptable crack widths for structures subject to deicing range from near 0 to 0.2 mm (0.001 to 0.008 in.).

When concrete cracks in a corrosive environment, embedded reinforcing steel can rapidly corrode. Cracks 0.05 mm (0.002 in.) or wider may allow infiltration of water and salts. Most cracks are aligned with the steel exposing large areas of the bar. A conservative approach of bonding or sealing all visible cracks is suggested when attempting to achieve long-term durability of structures subjected to deicers or those in a marine environment.

Transportation agencies should establish the cause of the cracking before selecting a repair method. Repair of cracking caused by expansion products from internal chemical reactions, cyclic freezing, or corrosion of embedded steel may not be effective. Removal and replacement of the affected concrete may be required. The following discussion pertains to the repair of typical early transverse cracking.

All visible cracks should be repaired after an age of 6 months to allow the initial shrinkage of the concrete to occur. Usual methods for crack repair in hardened concrete decks include epoxy injection, HMWM topical treatment, application of silane and siloxane sealers, and routing and sealing. According to ACI 224 (*56*), cracks as narrow as 0.05 mm (0.002 in.) may be repaired using epoxy injection, although the authors have found that such repair may not always be effective. HMWM resin and silane or siloxane sealers will work on cracks of even narrower widths. HMWM resins have been effective, when properly applied, in bonding and preventing infiltration of deicing solutions into both wide and hairline cracks. Penetrating sealers do not fill or bond the cracks but they make the sides of the narrow cracks water-repellent.

Final selection of a repair method and material should take into account ease of application, durability, life-cycle cost, available labor skills and equipment, and appearance of the final product. Reports on repair of cracks, such as those by ACI Committee 224 (*56*), contain relevant information that should be reviewed.

Epoxy injection generally consists of drilling holes at relatively close intervals along the cracks, sometimes installing entry ports, and injecting the two-component epoxy under pressure using specialized injection equipment. Pressure injection of cracks is labor intensive and time consuming. It is generally limited to decks containing a few large discrete cracks. Epoxy is the most commonly used resin for pressure injection applications, although other resins can be used. Detailed information on epoxy injection is included in ACI Committee 503R (*57*) and 224 (*56*) reports. Personnel experienced in epoxy injection should be used for this work.

Gravity feed techniques using HMWM resins were developed by Caltrans (*58*) for topical treatment of bridge decks that contain many fine cracks. The low viscosity HMWM resin (8 to 20 cps), readily flows into very fine cracks by gravity. Epoxy resins are also available to repair cracks by gravity feed. The epoxy resins typically do not penetrate fine cracks as deeply as the HMWM and they cure slower. However, epoxies may be the best selection for gravity feed of cracks wider than about 0.25 mm (0.010 in.) Trial areas are recommended.

The HMWM has a high solvent capacity that enables it to bond through lightly contaminated surfaces. Curing compounds and asphaltic materials should be removed before treatment because the resin will solvate them and thicken, causing poor final properties. The cracks must be dry, because water will prevent penetration and dilute the resin, also resulting in poor polymerization.

A metallic drier and peroxide catalyze the HMWM monomer to initiate polymerization. The resin is then swept, squeegeed, or sprayed on the cracked concrete at a rate of approximately 0.4 L/sq m (1 gal/100 sq ft). The resin flows into the cracks and polymerizes, filling and bonding them. Dry blasting sand should be broadcast into the resin, before the resin hardens, to improve skid resistance.

The performance of HMWM resins varies. The product selection should be based on satisfactory use in similar applications. A trial application is recommended for large jobs. Usually, the resin performs well when the concrete and air temperatures are between 7°C (45°F) and 32°C (90°F). Special formulations of HMWM resins are available for use during cold or hot weather. Sealing cracks with HMWM has achieved more than 11 years of satisfactory service in bridge decks in California subjected to freezing and thawing and deicers. The cracks remained filled, and maintenance was reduced.

HMWM resins are brittle, and traffic abrades them. They therefore do not function as a water-repellent. Because HMWM resins are compatible with silane sealers, HMWM resins can be used to fill and bond the cracks after silanes have been applied to seal the deck surface.

Many materials have been used over the years as coatings or sealers for concrete. Some have been effective and some have not. The first systematic study of sealers was done in 1979, and reported in *NCHRP Report No. 244* (*59*). This study found five categories of sealers effective: polyurethanes, methyl methacrylate, certain epoxy formulations, relatively low molecular weight siloxane oligomers, and silanes. Many polyurethanes currently sold have the limitation that they are not dependable when exposed to the ultraviolet (UV) rays of the sun. Epoxies, acrylics, and methacrylates are very effective sealers; however, at the viscosities normally used they do not penetrate into the cracked or uncracked concrete, but leave a continuous film on the surface and would be abraded off the deck by traffic. Also, they are not largely vapor permeable.

The effective penetration of silane and siloxane sealers is a result of their very small molecular size. These penetrating sealers penetrate into cracks and then infiltrate and coat the micropores and capillary structure of the concrete and crack surface. Penetrating sealers can penetrate cracks as deep as 100 mm (4 in.) and subsequent lateral penetration into the concrete within the cracked region of as much as 5 mm (3/16 in.) or more, depending on the density and finish of the concrete, the orientation of the surface, and the moisture content of the concrete.

Silanes and siloxanes are both derived from the silicone family. When catalyzed by moisture, these silicon materials react with the silica available in concrete to form a hydrophobic siloxane resin film that repels water without loss of vapor transmission properties. Both penetrate the concrete to some degree, the silanes being better in this aspect because of their smaller molecular size. They are typically 20 or 40 percent solutions in alcohol or petroleum distillates. Newer products include 100 percent solids silanes and water-dispersed silanes that can be used in areas requiring volatile organic content (VOC) compliance.

The effectiveness of penetrating sealers in preventing ingress of salt-laden water through cracks requires additional research. The maximum crack width that can be effectively sealed on a bridge deck subjected to high tire pressures is unknown and requires further research.

Because these penetrating sealers do not block the movement of air or water vapor, carbonation of the cracked concrete by ingress of carbon dioxide gas can still occur. In areas exposed to both moisture and carbonation, a penetrating silane material should be applied followed by a barrier coating such as an acrylic or epoxy coating or overlay.

Active cracks are unusual on bridge decks. However, some engineers feel that flexible repair materials are required because of crack movements related to thermal changes. Flexible sealants generally consist of urethanes, polysulfides, acrylics, silicones, or epoxies. Active or moving cracks must be treated as if they were control joints. Active cracks that must be made watertight cannot be easily repaired, because they change in width in response to changes in temperature or humidity. The difficulty is that, to provide the resilience in the finished repair required to maintain a seal, it is not enough to specify a material that has the proper elongation. A proper shape factor must also be provided. ACI 504 (*60*) contains an excellent discussion of shape factors. A material having an "elongation at break" of 100 percent, when tested according to ASTM D688 (*55*), cannot accommodate more than 3 or 4 percent elongation in a crack or a joint without the proper shape factor.

To provide a watertight seal in a crack, the crack must be routed to a width of at least 10 mm (3/8 in.) and bond must be prevented to the bottom of reservoir. This can normally be done easily with tape, or wax placed in the sawcut or routed channel. If the routed crack has a triangular shape, it must be partially backfilled with a soft, easily friable material, such as non-drying caulk, to achieve the required stretching length. If all this is done, with due regard to maintaining scrupulously clean concrete sides in the channel, the repair can be considered long-term. However, if exposed to sunlight, certain sealants may degrade in 4 to 8 years because of UV light exposure, water, and traffic.

CHAPTER 4

# CONCLUSIONS AND SUGGESTED RESEARCH

## CONCLUSIONS

Deck cracking is not confined to one geographic location and is of international concern. Of particular concern is the presence of cracks in newly constructed decks of high quality, low water-cement ratio concretes that also were cured with good moist curing procedures, such as observed in this project at the redecking of the Portland-Columbia Bridge. When environmental temperature and wind conditions make jobsite moist curing difficult, worse cracking generally occurs. Although plastic shrinkage cracks contribute to this overall deck problem, these plastic shrinkage cracks can be prevented by immediate use of water fogging and evaporation retarder films. Transportation agencies do not seem to recognize that the newer low water-cement ratio concretes with HRWRA, silica fume, and latex emulsions experience plastic shrinkage cracks, and that the accepted maximum evaporation rate of 1.0 $kg/m^2/hr$ (0.2 $lb/ft^2/hr$), which was indicated decades ago for normal concretes, may not be applicable to these newer concretes. These observations suggest that (1) bridge deck concrete specifications should include a bleeding test and (2) the allowable evaporation rate should be significantly lowered, as two transportation agencies have implemented.

Transverse cracks in newly constructed bridge decks are commonly observed above the reinforcing bars and are usually full-depth. The cracks are typically spaced between 1 and 3 m (3 and 10 ft) apart.

Cracks often shorten the service life and increase maintenance costs of concrete structures. Transverse cracking can promote spalling at crack intersections and accelerated corrosion of embedded reinforcing steel or other superstructure steel. Reduced durability of the concrete can also occur, because of increased saturation of the concrete.

Although researchers have written many articles and reports regarding the size of acceptable and unacceptable cracks, there is no consensus. These opinions may not pertain to bridge decks because they are based on research studies on cracks that are perpendicular to the reinforcing bars, and cracks in decks are commonly in-line with the transverse steel. Typical acceptable crack widths for structures subject to deicing range from near 0 to 0.2 mm (0.008 in.). Denmark, Japan, and Switzerland typically specify a maximum crack width of 0.2 mm (0.008 in.) on conventionally reinforced decks. Only two U.S. transportation agencies specify crack width limitations for new decks. One specifies 0.18 mm (0.007 in.), and the other specifies less than 15.2 m (50 ft) of cracks greater than 0.5 mm (0.020 in.) wide per 46.5 $m^2$ (500 sq ft) of deck. The shape and orientation of cracks with respect to the reinforcing bars were found to influence the extent of deterioration, with cracks in-line with the bars being worst. This common orientation of the crack directly over the transverse bar in a deck accelerates the steel corrosion. A conservative approach of bonding or sealing all visible cracks is suggested when attempting to achieve the most durable structure in an aggressive environment, due to this orientation problem.

The effects of concrete materials, design, and construction practices on concrete cracking are discussed. Concrete material factors important in reducing early cracking include low shrinkage, low modulus of elasticity, high creep, low heat of hydration, and selection of aggregates and concrete that provide a low cracking tendency. Other material factors helpful in reducing the risk of cracking include reducing the cement content, increasing the water-cement ratio, using shrinkage-compensating cement, and avoiding silica fume admixtures and other materials that produce very high early compressive strengths and modulus of elasticity values.

Because drying shrinkage of portland cement concrete is inevitable and is usually between 500 to 1000 $\mu\epsilon$, the use of other cementitious materials that have much less drying shrinkage could be pursued. Further testing of low shrinkage cementitious materials and admixtures should be performed.

The major design factors affecting cracking were related to restraint, specifically bridge type and girder type and size. Multispan continuous composite large steel-girder bridges are most susceptible to cracking because of additional restraint. Cast-in-place, post-tensioned bridges are the least likely to have deck cracking since the girders and the deck shrink together and the post-tensioning generally induces compressive stresses in the deck. Other design factors that may moderately contribute to early cracking include continuous spans, alignment of top and bottom transverse bars, and use of stay-in-place forms, which create shrinkage gradients in the deck.

The most important construction-related factors affecting transverse cracking involve weather and curing. Standardized curing procedures are not used. Transportation agencies observe more cracking when concrete is placed during lower humidities and higher evaporation rates. Wind breaks and sun shades are suggested during periods of high evaporation, and water fogging and evaporation retarder films are essential immediately following screeding. Immediate water fogging or application of evaporation retarding films should be performed on all deck placements regardless of evaporation rates or temperature. This is especially important when casting low water-cement ratio concrete mixtures. Standardized curing procedures should result in better quality and consistency, because finishers will know what procedures are required. Early wet curing should be done to reduce the evaporation of mix water and to cool the concrete. Heated concrete placed in cold weather is also susceptible to significant evaporation and surface moisture loss because of the concrete's high evaporation rates and the long set-time with winter conditions. Engineers and contractors should recognize the potential for very high evaporation of the mix water during cold weather castings and the required precautions. Evening or night deck construction should be considered.

This project developed a cracking-tendency test procedure to compare various concrete mixtures, curing, and environmental factors. The test involves the use of a steel tubing section to restrain a concrete ring cast around it. The steel pipe was instrumented with strain gages to detect first cracking, shown by an abrupt loss of compressive strain in the steel tubing. The concrete rings were also visually examined for cracking. The effects of many factors such as water-to-cement ratio, cement content, aggregate size and type, superplasticizer addition, silica fume, set accelerators and retarders, air entrainment, cyclic temperature, evaporation rate, curing, and shrinkage-compensating cement concretes were investigated. All mixtures under all the test conditions developed some type of cracking. The mixes that performed best in this test had essentially no slump and required excessive compaction to consolidate. Rings cast with Type K expansive cement cracked much later than the control concrete, with no clear cracks occurring. Aggregate type had the most dramatic effects on cracking, with crushed limestone concrete cracking latest and rounded river gravel concrete cracking earliest. Transportation agencies can use this test to evaluate local materials and to help select mixtures with the least tendency for cracking. Agencies should (1) evaluate local materials using the restrained ring test and (2) modify their standard mixes on the basis of the recommendations presented in this report to achieve acceptable concrete mix designs that have the lowest cracking tendency.

An instrumentation and monitoring system was designed and installed on the deck replacement of a steel-girder bridge that received 8 days of proper wet curing. Starting when the concrete was cast and continuing for several months after the deck cracked, the system measured strains, temperatures, and environmental factors, including rates of evaporation. Significant uniformly spaced cracking of the deck occurred between 26 to 49 days. However, review of the strains recorded by the strain gages on the embedded reinforcement suggest that cracking may have already occurred at lower depths but had not yet propagated to the upper surface or was not visible. Transportation agencies can instrument bridges to evaluate stresses.

This project derived systems of equations for the calculation of resultant tensile stresses in a composite reinforced concrete deck subjected to uniform and linear temperature changes and shrinkage. The equations accommodate multiple levels of embedded reinforcement, to include the effects of longitudinal bars and a stay-in-place metal deck. Data from the derived equations were compared with the actual measured strain behavior of the Portland-Columbia Bridge deck that showed visible deck cracking 26 to 49 days after casting. This comparison of elastic theory to actual measured strain and temperature behavior for this heavily instrumented bridge showed that during re-decking uniform strains and temperatures across the width or length of the bridge do not occur during the early life of the new deck. While a wide range of strains and temperatures were measured on the steel girders and within the reinforced concrete deck, the average strains correlated well with the calculated elastic strains. The elastic equations estimated with reasonable accuracy the thermal and shrinkage strains of the Portland-Columbia Bridge when standard concrete properties and shrinkage rates were applied. Therefore, the elastic equations can estimate stresses that develop in bridges from shrinkage and temperature changes. These same equations that were used to estimate the degree-of-restraint for different bridge geometries and concrete materials could be used by designers to evaluate different bridge designs and to reduce the deck restraint and related deck cracking. Further refinement of the equations and input of the real ever-changing concrete properties can result in accurate prediction of cracking in decks.

The parameter study for this project examined the stresses—in more than 18,000 bridge scenarios—caused by uniform and nonuniform shrinkage and temperatures in bridges, and determined how bridge geometry and material properties affect these stresses. Many conclusions have been reached.

Longitudinal tensile stresses in the concrete deck cause transverse deck cracking. The stresses that cause transverse deck cracking are largely caused by concrete shrinkage and changing bridge temperatures and, to a lesser extent, traffic. The parameter study examined the range of stresses caused by shrinkage and nonuniform temperatures in bridges, and determined how bridge geometry and material properties affect these stresses.

Deck shrinkage stresses are generally higher in a steel-girder bridge than in a concrete-girder bridge. Shrinkage stresses are typically lowest in a monolithic cast-in-place bridge, where both the deck and girders shrink similarly. A bridge deck supported by precast, prestressed girders may develop more or fewer shrinkage stresses than the monolithic concrete bridge, depending on the remaining shrinkage of the precast girders when the concrete deck is cast and the remaining creep of the precast, prestressed girder.

When the concrete deck shrinks relative to its girders in a simply supported span, 500 $\mu\epsilon$ uniform shrinkage of the concrete deck may cause tensile stresses as large as 9.65 MPa (1400 psi) in the steel-girder bridge and 12.4 MPa (1800 psi) in the concrete bridge, depending on geometry and material properties; maximum stresses from a linear shrinkage profile through the deck, such as with a steel SIP form, are slightly larger. These stresses can cause transverse cracking in bridge decks.

Additional shrinkage stresses develop over the interior supports of continuous-span bridges. These additional stresses are generally small in most bridges with steel girders. When concrete girders are used, these additional stresses are generally small when the bridge shrinks uniformly with depth. However, when differential shrinkage occurs between the concrete girders and the deck, total stresses over the interior support may reach nearly 13.8 MPa (2000 psi) for differential shrinkage of 500 $\mu\epsilon$, sometimes much larger than the stresses away from the supports.

Thermal stresses created from early hydration temperatures are largest in steel-girder bridges and in concrete bridges when the decks are cast separately from the girders. For most bridges, diurnal temperature changes produce larger thermal stresses than seasonal temperature changes. The parameter study analyses revealed that a linear temperature gradient in the deck, not a uniform temperature gradient, typically produces the largest deck stresses and greatest risk of transverse cracking. All of these stresses are sufficient to cause transverse deck cracking, especially over interior supports of a continuous-span structure. Seasonal temperature changes are small or negligible in concrete bridges because both deck and girders typically have similar thermal expansion rates.

Many factors affect shrinkage and thermal stresses. The primary factors include the concrete material itself, the geometry of the bridge, construction techniques, and the bridge environment. Shrinkage stresses are generally linearly proportional to the shrinkage of the concrete. Geometry affects shrinkage stresses less than material properties. Generally, larger deck stresses develop with (1) larger girders at a narrower spacing and (2) thinner decks.

## SUGGESTED RESEARCH

The project took a wide approach and investigated many variables affecting cracking of concrete. Although this work provided a great deal of insight, the results indicate that many areas require further study.

This comprehensive study shows conventionally reinforced concrete bridge decks that are composite with girders are likely to develop cracks during their early life, and may continue to crack during their long-term life. This cracking is dominated by the low tensile strain capacity of concrete. Several obvious approaches to reduce or eliminate cracking include the use of noncomposite bridge decks that reduce the degree of restraint mandated by composite bridge deck systems, and the use of prestressing to induce compressive stresses in bridge decks, either cast-in-place or precast.

Although noncomposite bridge decks are not commonly used because of economic considerations, their use will reduce deck cracking. The concept of prestressing a composite cast-in-place deck is complicated by the same restraint issue that creates the tensile stresses in decks and the subsequent cracks; that is, it is difficult to induce prestressing compressive stresses in the deck, because it is restrained from shortening in a composite deck girder bridge system. This shortcoming can be overcome by allowing the deck system to shorten during post-tensioning or pretensioning and then making the deck composite after the prestressing operation. This approach should also be pursued.

The bridge type that minimized the restraint issue while also utilizing prestressing advantages is the cast-in-place, post-tensioned concrete structure where slab, beams, and soffit are cast at the same time with subsequent post-tensioning. Whereas this bridge concept has generally been used for large bridge projects, its use should be explored with smaller bridges.

Because typical precast concrete-girder bridges have decks cast at a later time or steel girders that do not shrink in harmony with the deck, some degree of cracking is probably inevitable. For these decks with the cracks directly over the transverse bars, the use of highly corrosion-resistant bars can provide superior long-term performance and reduced maintenance. However, performance will be further enhanced if the cracks are repaired.

Since cracking is such a major problem with the most common bridge systems, standard repair procedures should be developed and incorporated into the contract specifications. Because most cracks appear before 6 to 12 months of age, the contract could include provisions for crack repair during the first year after construction, preferably prior to service and prior to significant contamination of the crack interior surfaces from environmental and deicer effects. Research should be undertaken to develop repair materials and procedures that will function under the difficult jobsite conditions, including penetrating sealer and crack filling materials.

A promising approach to reduce cracking was the use of expansive shrinkage-compensating concretes. However, the ring test used may not have accurately represented the restraint to initial expansion that exists in field structures. Studies should be done to develop a modified dual ring test that more accurately reflects the restraint against early expansion that exists in bridge decks.

Another promising approach to reducing cracking appears to lie in the type, size, shape, and surface characteristics of the coarse aggregates. Research is also needed on the use of low-shrinkage cements and other cementitious materials that have very low shrinkage. The use of polymer modifiers in low dosages may also yield durable low modulus concretes that have a low cracking tendency.

Further investigation of the effect of cement chemistry and fineness on early deck cracking is needed. Cements with various finenesses should be tested in the ring test and evaluated on field projects.

A comprehensive database of future restrained ring tests should be developed, including testing field mixes from different regions of the country, reflecting the aggregates and cements found in different regions. Correlation between the ring test results in the laboratory and ring tests and deck performance from jobsites on a variety of structure types is needed. Additional data will also offer insight on the effect of different admixture brands and dosages on cracking, because the work performed in this study only used one manufacturer of each admixture type and limited admixture dosage rates. Also, as the effects of the admixtures were tested on a limited range of concrete mixtures, further work should include different concrete proportions.

Direct comparisons of decks or deck sections built with normal-strength concrete and low early strength concrete should be done. Replacing cement with certain fly ashes or pozzolans may reduce early strength and modulus of elasticity, while increasing creep and improving deck performance. Early low modulus of elasticity values are desired. The ultimate design strength for this concrete should not be achieved until later ages such as 56 to 90 days.

Morning, afternoon, and evening placements should be evaluated. A series of simply supported spans with steel girders should be cast at various times of day and monitored for cracking, although evening castings are presently thought to be the most desirable.

The techniques used to monitor the Portland-Columbia Bridge deck replacement should be used on additional bridges to evaluate response and further refine analysis techniques. Comparing stresses of several bridge types will provide additional understanding of the complex interactions that occur in newly constructed bridge decks.

Transportation agencies can use the equations developed in this project to evaluate tensile stresses in decks for various bridge geometries and concrete properties. Further development of these equations should result in a reliable predictor of early bridge deck cracking. Agencies can then design structures and match concrete properties to eliminate or minimize deck cracking.

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# APPENDIXES A THROUGH H

## UNPUBLISHED MATERIAL

Appendixes A through H contained in the research agency's final report are not published herein. For a limited time, copies of that report entitled, "Transverse Cracking in Newly Constructed Bridge Decks—Appendices A-H," will be available on a loan basis or for purchase ($22.00) on request to NCHRP, Transportation Research Board, Box 289, Washington, D.C., 20055. The available appendixes are titled as follows:

Appendix A: Literature Review
Appendix B: Transportation Agency Surveys
Appendix C: Cracking-Tendency Test
Appendix D: Cracking-Tendency Test Procedure and Test Results
Appendix E: Field Instrumentation
Appendix F: Derivation of System of Linear Equations to Calculate Shrinkage and Thermal Stresses in Composite Beams with Deck Reinforcement
Appendix G: Elastic Theory and the Measured Behavior of the Portland-Columbia Bridge
Appendix H: Transverse Bridge Deck Cracking Parameter Study

# PROPOSED GUIDELINES FOR CONCRETE BRIDGE DECK MATERIALS AND CONSTRUCTION

## CONTENTS

## ACKNOWLEDGMENTS

These Guidelines were developed under NCHRP Project 12-37 by Wiss, Janney, Elstner Associates, Inc. Paul D. Krauss, P.E., and Ernest A. Rogalla, S.E., were Co-Principal Investigators. This work was done under the general supervision of Donald W. Pfeifer, S.E. David B. McDonald performed most of the literature research and Matthew R. Sherman conducted most of the ring shrinkage tests. Andrew E. N. Osborn supervised the field monitoring of the Portland-Columbia Bridge.

CHAPTER 1

# INTRODUCTION

## GENERAL

Many concrete bridge decks in the United States develop transverse cracks soon after construction. These cracks are full-depth and spaced 1 to 3 m (3 to 10 ft) apart along the length of the bridge. Transverse cracks may shorten the service life of a bridge and increase maintenance costs.

## WORK DONE

This project surveyed transportation agencies to learn how extensive transverse deck cracking is, how bridges are designed, and what construction methods are used to build bridge decks. Then, this project investigated the causes of early transverse cracking, and developed a test to evaluate and predict cracking tendency of concrete mixes. A detailed literature review was performed to learn how past research by others related to this project. Elastic equations were developed to calculate shrinkage and thermal stresses in a composite bridge with a reinforced deck. These equations were then used to analyze approximately 18,000 geometry and material combinations to determine combinations likely to produce high deck stresses and transverse cracking. Recommendations are presented to reduce transverse deck cracking. These Guidelines summarize the project findings, and the project final report provides more detailed information.

A survey was sent to all U.S. departments of transportation (DOTs) and several transportation agencies overseas to learn the extent of early transverse deck cracking. Fifty-two agencies from the United States and Canada responded to the survey. The respondents estimate that more than 100,000 bridges in the United States developed early transverse cracking, about half the bridges monitored by the respondents. Sixty-two percent of the agencies consider early transverse cracking a problem, 24 percent do not consider this a problem, and 14 percent have no opinion. Many states that do not consider transverse cracking a problem nonetheless reported extensive cracking. Fifteen percent believed that all of their decks have transverse cracks.

This project used three different methods to study the problem of transverse cracking: theoretical and analytical analysis, field instrumentation, and laboratory study. The analytical studies used conventional and finite element analyses to evaluate the causes of transverse cracking. The factors studied included concrete material properties, bridge geometry, and weather. Finite element analyses studied the combined effects of these factors on stresses that cause cracking in bridge decks.

The Portland-Columbia Bridge, located between Pennsylvania and New Jersey on Route 512, was instrumented and monitored during its deck replacement in 1992. The monitoring started a few hours before the concrete was placed and continued for several months after the deck cracked. These collected data provide important insight into the detailed thermal and shrinkage behavior of a steel-girder bridge during construction. This information was later used to compare measured behavior to theoretical behavior. The instrumentation techniques discussed in the final report may be helpful to agencies interested in instrumenting a deck installation.

This project determined that, for most bridges, concrete properties affect cracking more than any other factor. As such, a test procedure was developed to measure the cracking tendency of different concretes. In brief, the test involves casting a concrete ring against a steel inner ring that restrains the shrinking concrete and usually causes cracking. Strains are measured in the steel ring to monitor strains in the concrete and estimate stresses. The steel ring restrains the concrete ring similar to the way composite bridge girders restrain a concrete deck. This test combines the complex interaction of the varied and changing concrete properties that affect cracking, including shrinkage, modulus of elasticity, tensile strength, and creep. Concretes of various aggregate types, mix proportions, admixtures, curing, and environmental conditions were tested. AASHTO should adopt this test procedure, so that transportation agencies can test and develop concrete mixes that resist transverse cracking. This test can be used to prequalify concrete mixes or can be incorporated into contract specifications.

These Guidelines discuss the major findings of the research, and identify and rank the factors and combinations of factors that cause transverse cracking in new bridge decks. These factors are categorized by design, materials, and construction. The intent of these Guidelines is to help agencies reduce or eliminate transverse cracking. Specific actions to reduce deck cracking are presented. Careful consideration of the impact of the recommendations may be needed to avoid adversely impacting other design or durability concerns.

## AASHTO SPECIFICATIONS

Many engineers perceived that transverse bridge deck cracking increased in the mid-1970s, when AASHTO made major changes to its concrete specifications (*1*), AASHTO periodically changed its specifications to produce stronger and less permeable concrete. In 1974, the AASHTO specifications increased the minimum compressive strength, specified a low water-cement ratio, and increased the minimum cement content.

Modern cements have also worsened deck cracking because of their increased fineness, higher sulfate and alkali content, and higher heat of hydration. The finer modern cements reach high early strengths and typically have 1-day compressive strengths near 45 percent of the 28-day strengths. Cement manufactured in the mid-1940s had 1-day compressive strengths of only 11 percent of the 28-day strengths. The high early strength and modulus combined with higher heat of hydration result in increased locked-in stresses during very early deck ages.

CHAPTER 2

# CRACKING AND DURABILITY

## CRACK FORMATION

Concrete cracking, after its final set, is classified into two categories: microcracking and macrocracking. Microcracking precedes macrocracking.

### Microcracks

All concretes develop microcracks that are visible only with magnification and typically are narrower than 0.025 mm (0.001 in.). Microcracks are randomly oriented and located at the mortar and coarse aggregate interface. These interfacial bond cracks begin when the concrete is hydrating and shrinking, before external loads are applied. Stresses develop at the paste-aggregate interface from the volume change of the cement paste and the restraint provided by the aggregate. Microcracks are discontinuous and very narrow, and do not affect serviceability of a bridge deck.

### Macrocracks

Fractures visible with little or no magnification are macrocracks. Large macrocracks in a bridge deck should to be filled to prevent the penetration of water or aggressive solutions into the concrete and to re-establish structural integrity when necessary. Macrocracks initiate in the weakest section of the member, usually at random planes. When concrete adjacent to reinforcement cracks, some slip between the concrete and steel occurs from the stiffness and strength incompatibility of the concrete and steel and the associated bond. In beams with mature concrete, reinforcement stresses during this phase are often greater than 96 MPa (14,000 psi) (*2*).

Concrete is in its final stage of cracking when additional cracks do not form, but cracks widen instead. At this stage, slippage of the reinforcement continues, and reinforcement stresses in mature concrete often exceed 200 MPa (30,000 psi) (*2*). Crack widths are usually narrower at the reinforcement and wider at the outer surface.

When reinforced concrete cracks, slippage occurs between the reinforcement and the concrete. The tensile stress carried by the concrete before fracture is then transferred to the reinforcement within the cracked region. The concrete stresses on both faces of the fracture dissipate, and decreasing slippage between the concrete and reinforcement extends into the uncracked section on each side of the fracture. The tensile force in the reinforcement is gradually transferred to the concrete within this bond-slip distance, and at the end of the bond-slip, the tensile stresses have been fully transferred to the concrete. Strain compatibility between the concrete and reinforcement does not exist within the bond-slip distance.

Within the bond-slip length, the concrete tensile stresses are typically less than the tensile strength of the concrete. As a result, no additional cracking occurs within the bond-slip distance. This bond-slip distance determines the minimum crack spacing that can develop.

## CRACK CHARACTERISTICS

Many researchers have studied the effects of crack length and width on concrete performance, without reaching a consensus. Transverse deck cracking can cause accelerated corrosion of reinforcing steel, deterioration and leaching of concrete, accelerated damage to structural members and components beneath the deck, and appearance concerns. The orientation and shape of the crack with respect to the reinforcing affect deterioration. Once cracking occurs in concrete bridge decks, deterioration may accelerate. Corroding reinforcing steel may spall concrete. Traffic stresses and vibration on continuous-span structures can worsen cracking. Resulting deck repairs can be very costly and disruptive to traffic.

Full-depth deck cracking is generally the most severe form of deck cracking, because full-depth deck cracks are often wide and allow water and deicing chemicals to infiltrate the concrete rapidly. A wide surface crack that quickly narrows with depth may not be as detrimental as a narrower surface crack that has parallel sides. The crack width at reinforcement is affected by cover, reinforcement stress, concrete creep, reinforcement ratio, bar diameter and arrangement, and the profile of deck stresses.

Crack and reinforcement alignment affect reinforcement corrosion. When the crack is aligned directly over the reinforcement, corrosion can continue over the length of the bar at an increased rate. On the other hand, when a crack is perpendicular to a reinforcing bar, the crack exposes only a small section of the bar and corrosion will be slower. Most transverse deck cracks align with the transverse reinforcing and increase the potential for corrosion along the length of this steel.

## ACCEPTABLE CRACK WIDTHS

Researchers disagree about how wide a crack in a bridge deck can be without significantly affecting performance. Large crack widths may be aesthetically unacceptable. Lighting conditions, viewing distance, and crack length and width affect the perceived appearance of a crack and its width. Some agencies (*3*) have limited acceptable crack widths to 0.3 mm (0.010 in.) or less for aesthetic reasons, but this value is subjective.

Denmark, Japan, and Switzerland typically limit crack widths on conventionally reinforced decks to 0.2 mm (0.008 in.). Only two DOTs limit crack widths; one limits crack widths to 0.18 mm (0.007 in.), and the other requires less than 15.2 m (50 ft) of cracks greater than 0.5 mm (0.020 in.) wide per 46.5 $m^2$ (500 sq ft) of deck.

Cracks allow the ingress of chlorides and moisture. ACI Committee 224 (*4*) suggests that a surface crack width of 0.18 mm (0.007 in.) is tolerable for structures subjected to deicing chemicals and 0.15 mm (0.006 in.) is tolerable for structures subjected to sea water and wetting and drying. However, many others have recommended narrower crack widths. Crack widths of 0.2 mm (0.008 in.) in concrete exposed to 10 years of sea coast exposure (*5*) caused the concrete to absorb large amounts of chlorides, which resulted in severe corrosion of the steel reinforcement. The authors investigated many cracked decks and found water leakage through cracks as narrow as 0.05 mm (0.002 in.), with accelerated corrosion of embedded reinforcing steel and supporting girders at the crack.

Cracks may also reduce the durability of epoxy-coated reinforcement (*6*). While the corrosion resistance of reinforcing steel is greatly enhanced by the epoxy coating, corrosion can occur where breaks or other defects in the coating are present (*7,8*).

When decks are in an aggressive environment and are subjected to deicers or sea spray, all visible cracks should be filled or sealed to maintain durability. This repair will delay chloride penetration to the steel and reduce the corrosion at cracks. Usual methods for crack repair include epoxy injection, high molecular weight methacrylate (HMWM) topical treatment, silane and siloxane sealers, and routing and sealing. Chapter 11 of this document describes these repair methods.

## CARBONATION

Many perceive cracking mainly to affect structures subjected to deicing chemical or sea water exposure. However, carbonation of concrete at cracks may also increase corrosion of the embedded steel, either with the aid of chloride or without. Concrete carbonates when atmospheric carbon dioxide ($CO_2$) in the crack reacts with its moisture and hydration products. This decreases the pH of the pore liquid from approximately 13 to less than 9. With carbonation, the alkalinity of the concrete is no longer present to protect the steel against corrosion, and the corrosion system can become active. Cracks allow direct ingress of $CO_2$ into the structure, which can accelerate corrosion. Carbonation alone does not cause rapid corrosion, but this corrosion eventually can cause the same damage that chloride-instigated electrochemical corrosion does. Carbonation of deck concrete can also contribute to cracking because it causes irreversible shrinkage, which can be as large as drying shrinkage.

CHAPTER 3

# CRACKING-TENDENCY TEST

## INTRODUCTION

It is important to realize that cracking and shrinkage are not synonymous. Researchers (*10,11,12,13,14*) have used many different tests to investigate concrete shrinkage and cracking. These tests have included restrained linear prisms and thick restrained concrete rings. The restrained ring test has successfully predicted when concrete is susceptible to cracking; it was selected for this project, with some modification.

The major advantage of the restrained ring test is that it accounts for all of the material factors that influence shrinkage cracking from the time of casting. It does not require complex calculations or assumptions of early concrete behavior. It simultaneously considers stress development, dimensional changes, and creep at early ages. Usually, cracks are readily visible with the naked eye. The test is simple to execute, and the apparatus is inexpensive. Most important, stresses developed in the restrained test samples closely simulate those developed by real structural systems.

Finite element analyses examined theoretical shrinkage stresses in the steel inner-ring and the restrained concrete outer-ring, for various steel and concrete radii and longitudinal thicknesses and for various concrete properties. These analyses revealed that for steel-ring radial thicknesses between 13 and 25 mm (½ and 1 in.), concrete shrinkage stresses and cracking-tendency are not significantly different, but stresses in the steel can substantially increase as the steel thickness decreases. The analyses also showed that larger concrete stresses develop with larger diameters, especially with elastically stiffer concretes. Stresses in the concrete decrease as the thickness of the concrete ring increases. Therefore, the highest concrete stresses occur with restrained rings having a large diameter and thin concrete cover.

This project tested concretes under standard environmental conditions to determine the concrete mix design factors that influence cracking. This project examined cracking effects of cement content, water-cement ratio, cement type, silica fume addition, fly ash addition, aggregate type, superplasticizers, certain chemical admixtures, and entrained air. The effects of curing period, temperature, evaporation rate, casting time, and insulation were also investigated on specimens using selected concrete mixtures with high- and low-cracking tendencies.

## GENERAL DESCRIPTION OF CRACKING-TENDENCY TEST PROCEDURE

In the test, a concrete ring is cast around a steel ring instrumented with strain gages. The proposed test procedure is described in the section following these Guidelines. The strain accumulation and the length of time before cracking occurs indicate the cracking tendency of the concrete. Concretes that create less strain on the steel ring and take longer to crack have a lower cracking tendency.

Steel rings with an outside diameter of 305 mm (12 in.), a radial thickness of 19 mm (¾ in.), and a height of 152 mm (6 in.) were used in the test. These rings were custom-machined for this project and are more expensive than off-the-shelf pipe sections. To reduce the cost of steel rings for widespread field or laboratory use, structural steel pipe conforming to ASTM A501 or A53 (*15*) may be substituted without significantly affecting concrete stresses and cracking. The dimensions of a "12-in. (305-mm) extra strong pipe" are similar to the steel rings used in this project; this substitute has an outside diameter of 324 mm (12¾ in.), and a wall thickness of 13 mm (½ in.).

As discussed, the diameter of the steel ring affects the concrete shrinkage restraint, with larger diameter rings providing larger restraint. The 305-mm (12-in.) diameter of the steel rings used for this project approximates the worst-case shrinkage restraint on a deck, such as that typically provided by large steel girders. The test rings cast with the same concrete used to cast the bridge deck of the Portland-Columbia Bridge cracked at about the same time as the bridge deck did, indicating that the ring test and ring diameter approximated the restraint provided by the large steel-plate girders. Because one concrete mix may be used on many bridge types, the 305-mm (12-in.) ring is generally recommended to test concrete mixes. Future research should further validate the correlation of ring test results with bridge deck performance.

Except where noted, the casting, curing, and testing regime followed for the rings were as follows. Two concrete rings, five 100 × 200 mm (4 × 8 in.) cylinders, and two 75 × 75 × 280 mm (3 × 3 × 11 in.) free-shrinkage specimens were cast for each batch. The concrete was batched and cast at room temperature, using ingredients stored at room temperature. The air content, slump, and unit weight were measured immediately before casting.

The strength and free-shrinkage specimens were cast according to AASHTO T126 and T160 (*16*), respectively. The concrete rings were cast in three lifts, with rodding and spading of the sides to ensure proper compaction. The rings were moved to their final testing location and connected to strain-gage monitoring equipment immediately after casting. Once bleed water no longer appeared on the concrete surfaces, the central hold-down of the steel ring was loosened and the concrete was covered with water-saturated burlap. All specimens were removed from their forms approximately 24 hrs after casting. The free-shrinkage specimens were immediately placed in a 22°C (72°F), 50 percent relative-humidity room and measured. The evaporation rate in the controlled environment was approximately 0.15 kg/m$^3$/hr (0.03 lb/ft$^2$/hr). The cylinders were cured in lime-saturated water. The ring forms were removed and the bottom of the concrete rings was loosened from the lower form by applying gentle side pressure to the ring while lightly tapping the lower form with a rubber mallet, if necessary. The bottom form remained in place and the top surface of the concrete ring was then covered in a double-layer of polyethylene or rubber to prevent moisture loss and drying from the top and bottom of the ring.

The cylinder compressive strengths were measured at 7 and 28 days. The strains in the steel rings were measured hourly using computer-controlled data acquisition equipment. The ring strains were periodically analyzed, and the rings were carefully examined when the strains significantly changed. Typical initial crack widths were 0.05 mm (0.002 in.) or wider. After a ring cracked, the initial crack width was measured, using a visual crack comparator, and the ring was monitored for at least one additional week. After this week, the crack width was measured, the cracking was photographed, and the concrete was removed so that the steel inner-ring could be reused.

## OVERVIEW OF CRACKING-TENDENCY TEST RESULTS

This project investigated the cracking tendency of 39 concrete mixes. Test results are summarized in Table 1. Except where noted, all of the mixtures had the same granitic gravel and natural river sand from Eau Claire, Wisconsin, and cement from the same supplier. All mixes had the same mortar (cement + sand + water) volume throughout the testing. The following is a brief summary of the testing; additional details are presented in the project final report.

### Water-Cement Ratio and Cement Content

The first group of tests investigated the effect of water-cement ratio and cement content on cracking. The test results of the 11 mixes are shown in Tables 2 and 3. The mixes that performed best had low cement and water contents and essentially no slump. They required

**TABLE 1 Cracking-tendency test results sorted by average strain at cracking divided by average age at cracking**

| Mix No. | Description | Cement content, kg/m$^3$ | Cement content, (lb/yd$^3$) | w/c | Average cracking age, days | Steel strain at cracking, average | Avg. strain over avg. time, $\mu\epsilon$/day | Crack width day 5, mm | Crack width day 5, (in.) |
|---|---|---|---|---|---|---|---|---|---|
| 17 | Limestone | 390 | 658 | 0.44 | 100.0* | 72.5 | 0.73 | 0.03 | 0.001 |
| 23 | Type K | 390 | 658 | 0.44 | 100.0* | 75.0 | 0.75 | 0.03 | 0.001 |
| 28 | 60-day cure, high tendency | 501 | 846 | 0.35 | 81.0 | 62.5 | 0.77 | 0.10 | 0.004 |
| 31 | 60-day, low tendency | 278 | 470 | 0.50 | 84.1 | 65.0 | 0.77 | 0.00 | 0.000 |
| 8 | | 278 | 470 | 0.35 | 53.0 | 80.0 | 1.51 | 0.08 | 0.003 |
| 20 | Lightweight | 390 | 658 | 0.44 | 23.0 | 40.0 | 1.74 | 0.05 | 0.002 |
| 27 | Expansive additive | 390 | 658 | 0.44 | 36.5 | 67.5 | 1.85 | 0.23 | 0.009 |
| 21 | Trap rock | 390 | 658 | 0.44 | 32.3 | 61.0 | 1.89 | 0.10 | 0.004 |
| 4 | | 278 | 470 | 0.30 | 40.5 | 91.0 | 2.25 | 0.05 | 0.002 |
| 25 | 28 percent fly ash | 390 | 658 | 0.44 | 24.5 | 57.5 | 2.35 | 0.08 | 0.003 |
| 29 | No cure, low tendency | 278 | 470 | 0.50 | 22.3 | 55.0 | 2.47 | 0.08 | 0.003 |
| 3 | Low tendency control | 278 | 470 | 0.50 | 25.0 | 62.5 | 2.50 | 0.05 | 0.002 |
| 32 | Insulated curing, low tendency | 278 | 470 | 0.50 | 25.8 | 66.5 | 2.58 | 0.05 | 0.002 |
| 12 | No air | 390 | 658 | 0.44 | 21.3 | 57.0 | 2.68 | 0.08 | 0.003 |
| 19 | Accelerator | 390 | 658 | 0.44 | 17.5 | 50.0 | 2.86 | 0.13 | 0.005 |
| 7 | | 390 | 658 | 0.50 | 18.5 | 54.0 | 2.93 | 0.25 | 0.010 |
| 9 | | 278 | 470 | 0.44 | 19.8 | 58.5 | 2.95 | 0.05 | 0.002 |
| 22 | 13-mm (½-in.) Eau Claire | 390 | 658 | 0.44 | 20.7 | 61.5 | 2.97 | 0.18 | 0.007 |
| 5 | | 390 | 658 | 0.44 | 20.2 | 62.0 | 3.07 | 0.18 | 0.007 |
| 11 | | 501 | 846 | 0.44 | 20.1 | 62.5 | 3.12 | 0.18 | 0.007 |
| 13 | HRWRA | 278 | 470 | 0.35 | 25.5 | 82.5 | 3.24 | 0.05 | 0.002 |
| 18 | Retarder | 390 | 658 | 0.44 | 18.5 | 60.0 | 3.24 | 0.05 | 0.002 |
| 14 | HRWRA | 390 | 658 | 0.35 | 22.2 | 79.5 | 3.59 | 0.10 | 0.004 |
| 34 | High evaporation rate, low tendency | 278 | 470 | 0.50 | 16.1 | 62.5 | 3.88 | 0.05 | 0.002 |
| 6 | | 390 | 658 | 0.35 | 17.6 | 72.5 | 4.12 | 0.13 | 0.005 |
| 16 | Silica fume w/HRWRA | 390 | 658 | 0.35 | 16.3 | 67.5 | 4.14 | 0.08 | 0.003 |
| 1 | | 390 | 658 | 0.40 | 17.0 | 71.0 | 4.18 | 0.10 | 0.004 |
| 15 | Silica fume w/o HRWRA | 390 | 658 | 0.35 | 12.5 | 67.5 | 5.42 | 0.08 | 0.003 |
| 33 | Insulated curing, high tendency | 501 | 846 | 0.35 | 13.2 | 77.5 | 5.88 | 0.13 | 0.005 |
| 10 | High tendency control | 501 | 846 | 0.35 | 11.7 | 75.0 | 6.41 | 0.18 | 0.007 |
| 26 | No cure, high tendency | 501 | 846 | 0.35 | 9.6 | 63.5 | 6.64 | 0.08 | 0.003 |
| 30 | 6-hr delay in curing, high tendency | 501 | 846 | 0.35 | 11.5 | 80.0 | 6.97 | 0.18 | 0.007 |
| 2 | | 501 | 846 | 0.30 | 10.5 | 78.0 | 7.43 | 0.18 | 0.007 |
| 35 | High evaporation rate, high tendency | 501 | 846 | 0.35 | 6.8 | 69.0 | 10.15 | | |

* Assumed value, no cracking

**TABLE 2 Results of mixture proportion tests**

| Mix No. | Cement factor kg/m³ | (lb/yd³) | Water-cement ratio | Ring 1 crack age (days) | Ring 2 crack age (days) | Average time-to-cracking (days) | Free shrinkage at cracking ($\mu\epsilon$) | 200-day free shrinkage ($\mu\epsilon$) |
|---|---|---|---|---|---|---|---|---|
| 1 | 390 | (658) | 0.40 | 15 | 19 | 17 | 363 | 681 |
| 2 | 501 | (846) | 0.30 | 28 | 13 | 10.5 | 350 | 688 |
| 3 | 278 | (470) | 0.50 | 27 | 23 | 25 | 321 | 559 |
| 4 | 278 | (470) | 0.30 | 37.7 | 43.2 | 40.5 | 400 | 522 |
| 5 | 390 | (658) | 0.44 | 21.6 | 18.8 | 20.2 | 395 | 723 |
| 6 | 390 | (658) | 0.35 | 18.8 | 16.4 | 17.6 | 355 | 725 |
| 7 | 390 | (658) | 0.50 | 17.1 | 19.8 | 18.5 | 400 | 820 |
| 8 | 278 | (470) | 0.35 | 45 | 61 | 53 | 447 | 595 |
| 9 | 278 | (470) | 0.44 | 18.4 | 21.2 | 19.8 | 347 | 630 |
| 10 | 501 | (846) | 0.35 | 12.7 | 10.7 | 11.7 | 432 | 885 |
| 11 | 501 | (846) | 0.44 | 10 | 21.7 | 20.1 | 521 | 988 |

excessive compactive effort to consolidate and are not practical mixes for field placement. Both mixes contained very low paste volumes, because both the cement and the water contents were low. For the remaining mixes, the cracking tendency usually decreased as the cement content decreased and the water-cement ratio increased.

All concrete with a cement content of 390 kg/m³ (658 lb/yd³) cracked at essentially the same age (between 17 and 20 days), yet slump ranged from 37 to 254 mm (1½ to 10 in.) and water contents ranged from 137 to 195 kg/m³ (230 to 320 lbs/yd³). These data suggest that cracking was not dramatically affected by water content or water-cement ratio from 0.35 to 0.50 for these moderately high cement-content mixes. All three AASHTO-quality 0.44 water-cement ratio mixtures with three cement contents from 280 to 500 kg/m³ (470 to 846 lb/yd³) cracked at an age of 20 days. However, the data for the 0.30, 0.35, and 0.50 water-cement ratio mixtures show that increasing the cement content causes quicker cracking.

Sometimes but not always, concretes with a higher total paste content had higher cracking tendencies. Free-shrinkage was directly proportional to the paste content. The relationship between paste content and free-shrinkage was more apparent than that between paste content or free shrinkage and cracking tendency. This reflects the difficulty in predicting cracking because of the complex interaction of shrinkage, strength and moduli development, and early creep.

## Aggregate Type

Aggregate type significantly affected when the concrete rings cracked. This project investigated four aggregate types. The aggregate types and sizes by ASTM C33, *Standard*

**TABLE 3 Average time-to-cracking (days)**

| Cement factor kg/m³ | (lb/yd³) | Water-cement ratio 0.30 | 0.35 | 0.40 | 0.44 | 0.50 |
|---|---|---|---|---|---|---|
| 278 | (470) | 40.5 | 53.0 | — | 19.8 | 25.0 |
| 390 | (658) | — | 17.6 | 17.0 | 20.2 | 18.5 |
| 501 | (846) | 10.5 | 11.7 | — | 20.1 | — |

*Specification for Concrete Aggregates*, were No. 8 lightweight expanded shale, No. 56 crushed limestone, No. 8 trap rock, and No. 7 Eau Clair river gravel.

The most notable behavior was observed in the concrete using the No. 56 crushed limestone, which had a moderately high modulus of elasticity. The rings cast with this material did not exhibit a single distinct crack, but showed minor surface cracking. The surface cracking was only 25 mm (1 in.) deep and did not progress to the central steel ring. The limestone rings did not exhibit an abrupt decrease in compressive strain but showed a slow gradual loss of strain. The tests were discontinued after approximately 280 days when the ring strains became nearly constant.

The lightweight aggregate samples also exhibited notable behavior. In contrast to the limestone rings, the lightweight concrete rings developed large external cracks with no loss in steel ring strain. Instead, only a change in the slope of the strain-time curve was noted. This may have been due to the extremely low modulus of elasticity of the lightweight concrete that reduced stresses. When the cracking formed, the low stored energy was only partially dissipated in cracking and absorbed through aggregate interlock across the crack and by the friction developed between the concrete and steel because of the specimen geometry.

The specimens with trap rock aggregate had elastic moduli similar to the control concrete, but they cracked much later. The aggregate size did not appear to be a factor within the small range tested. Besides composition, the only other major difference between the aggregates was the aggregate shape; the limestone, lightweight, and trap rock aggregates were angular and the Eau Clair aggregate was well-rounded.

### Shrinkage-Compensating Cement

Tests were also done to investigate the effect of shrinkage-compensating materials. Two mixes were produced, one with Type K expansive cement and one with a commercially available expansive additive. The concrete with Type K cement did not develop distinct cracks. The concrete with the expansive additive cracked much later than the control concrete. The tests indicate an improved performance using expansive cements, although field performance has been mixed. The ring test did not restrain the concrete from early expansion and may not accurately reflect the performance of a bridge deck, suggesting the need for further testing.

### Fly Ash

A mix with 28 percent of the portland cement replaced with a Type F fly ash was tested. The mix had the same cement and water-cementitious ratio as the control concrete. The specimens with fly ash cracked only slightly later (4.3 days) than the control specimens.

### Chemical Admixtures

Chemical admixtures, including high-range water reducing admixtures (HRWRAs), set accelerators and retarders, and air-entraining admixtures were investigated. Air entrainment did not significantly affect the time-to-cracking. Except for the no-slump concrete, the HRWRAs delayed cracking 3.8 days when used with silica fume and 4.6 days when used without silica fume. On average, the concretes with accelerators or retarders cracked about 2 days earlier than the control specimens, but individual cracking times varied considerably.

### Silica Fume

The effect of silica fume was investigated using two mixes, one with a HRWRA and one without. The mixes containing silica fume cracked 5 to 6 days earlier than the companion mixes without silica fume. The free shrinkage of the silica fume concrete was similar to the control concrete. This again shows the complex interaction between strength, modulus, creep, and shrinkage on cracking.

### Curing

Curing was investigated with seven concrete mixes subjected to curing regimes including no curing, 6-hr delayed curing, 60-day wet curing, and thermally insulated curing. The different curing conditions were tested on high- and low-cracking-tendency concrete mixes selected from the mixes that investigated the effect of mix proportions. The low-cracking-tendency mix contained 278 kg/m$^3$ (470 lb/yd$^3$) cement and a water-cement ratio of 0.50. The high-cracking-tendency mix contained 501 kg/m$^3$ (846 lb/yd$^3$) cement and a water-cement ratio of 0.35.

The rings that were not cured (they were stripped out of the forms immediately after reaching final set) cracked quicker than the control specimens for both the high- and low-cracking-tendency mixes. No difference was noted for the 6-hr delayed curing concrete, although the geometry of the test specimens effectively prevented evaporation from the concrete test surface during the period before initial set.

In comparison to the control mixes, the 60-day wet curing delayed cracking of the high-cracking-tendency mixes by about 9 days, but showed mixed results for the low-cracking-tendency mix. Insulating the concrete to slow heat loss after it reached peak hydration temperature was inconclusive, with a large scatter in the test results.

### Evaporation Rate

This project tested the effect of evaporation rates on the high- and low-cracking-tendency mixes described above. The rings were cured in the forms under saturated burlap for 24 hrs before the forms were stripped. This precluded the development of plastic shrinkage cracks in the surface of the concrete. The rings in the high-evaporation-rate chamber cracked much earlier than the companion rings, confirming previous findings that linked high evaporation rates to earlier cracking. This earlier cracking occurred because strength development is not very dependent on the evaporation rate and the high-evaporation-rate specimens experienced larger shrinkage stresses without corresponding increases in strength.

### Casting Time-of-Day

The last group of tests investigated casting time. One set of mixes was cast in a simulated morning placement, and the other set was cast in a simulated evening placement. Each set consisted of two rings each of the low- and high-cracking-tendency mixes. The concrete rings were cast at room temperature, using materials stored at room temperature, and placed into the cyclic temperature chamber immediately after casting. For high-cracking-tendency concretes, the morning-cast specimens cracked 5 days earlier than the evening-cast specimens. On average, the low-cracking-tendency specimens cast in the morning cracked 13 days earlier than those cast in the evening.

CHAPTER 4

# STRESSES AND TRANSVERSE DECK CRACKING

## INTRODUCTION

Concrete bridge decks develop transverse cracks when longitudinal tensile stresses in the deck exceed the tensile strength of the concrete. These tensile stresses are caused by temperature changes in the concrete, concrete shrinkage, and bending from self-weight and traffic. A combination of shrinkage and thermal stresses causes most transverse bridge deck cracking. Transverse cracking in simply supported bridges and in continuous-span bridges away from interior supports is caused entirely by thermal and shrinkage stresses. In continuous-span bridges near interior supports, gravity-load stresses can increase the risk or severity of transverse cracking. Because many cracks occur before bridges are open to traffic, traffic loads are not considered a significant cause of early deck cracking.

Shrinkage and temperature stresses develop in all bridge decks, because the girders restrain the natural thermal and shrinkage movement of the deck. When the deck and girders consist of different materials (steel and concrete, or different concretes) with different thermal expansion rates, even a uniform temperature change will cause stresses as the different materials expand differently but cannot where they are attached. Temperatures in a bridge are rarely uniform or linearly distributed, and shrinkage is nonlinearly distributed. Nonlinear shrinkage and temperature changes cause stress, even without restraint.

Many factors affect shrinkage and thermal stresses. Geographic location affects these stresses, because the environment affects early hydration temperatures, drying shrinkage, and final temperature cycling. Material properties and bridge geometry also affect shrinkage and thermal stresses.

## RESTRAINT

Unrestrained concrete will expand when it is heated and contract when it cools. Unrestrained concrete also shrinks as it dries. Strain describes these thermal and shrinkage movements.

Strain by itself does not necessarily cause stress (necessary for cracking). When concrete undergoes a uniform or linearly distributed shrinkage or temperature change, it will not develop stresses if it is unrestrained. However, if restrained, the force or pressure restraining the concrete causes stress. Concrete restraint can be internal or external. Nonlinear shrinkage and temperature changes in concrete are restrained by the concrete itself and stresses develop.

Because most decks are composite with their supporting girders, or if not there is considerable friction reducing sliding, the girders restrain deck strains when they do not have temperature or shrinkage strains identical to the deck. This restraint is usually worst with steel girders, because steel girders do not shrink and steel usually has a different coefficient of thermal expansion. To a lesser extent, embedded reinforcement in the deck also restrains the deck against shrinkage and against thermal movements when the steel has a different coefficient of thermal expansion than the concrete. The girder and reinforcement restraint cause stresses in the concrete deck.

## TEMPERATURES AND THERMAL STRESSES

Most bridge decks experience temperature changes. These temperature changes cause stresses that cause transverse deck cracking. The first large stresses in a bridge deck develop during early hydration of the concrete deck. Later, temperature cycling causes additional stresses.

### Hydration Temperatures and Stresses

The first significant thermal cycle of a bridge develops during early hydration of the concrete. As fresh concrete hydrates and gains strength, the exothermic chemical reaction of the cement paste liberates heat and the temperature of the concrete rises.

During this time, the concrete is hydrating and generating substantial heat; this heat gain is larger with thicker members. Then, temperatures drop to match surrounding air or structure temperatures. While concrete is fluid, it can adjust to changing temperatures without developing stresses; however, after hardening, temperature changes cause stresses. Early thermal stresses in the concrete deck are highest if the deck concrete hardens when it is at its warmest temperature, locking in a stress-free temperature that is significantly warmer than the girders, followed by cooling that is restrained by the girders.

Peak temperatures typically occur within the first 24 hours after casting, after which concrete temperatures fall. Warmer concrete delivery, more cement, finer cement, thicker concrete sections, better insulated forms, warmer weather, more solar radiation, and warmer abutting structures increase early peak concrete temperatures and associated thermal stresses.

When a deck is cast monolithically with concrete girders, the concrete deck will reach warmer temperatures faster than when the deck is cast after the girders, because the girders are also generating heat instead of transferring heat from the deck. However, depending on bridge geometry and the environment, monolithic casting may increase or decrease stresses. Stresses from a monolithic pour can be lower when the girders are thin and heat and cool similarly to the deck; stresses can be higher when the girders are thick, keeping the deck above the girder warm while deck areas between the girders cool. An advantage of monolithically casting the girders and deck is that both will have simultaneous shrinkage, thereby, reducing shrinkage stresses.

The deck of the monitored Portland-Columbia Bridge was cast on large steel girders and is only 200-mm (8-in.) thick, yet temperatures increased and then decreased approximately 27°C (50°F) during the first 3 days. Thicker concrete sections dissipate hydration heat slower, and temperature can increase as much as 56°C (100°F) in thicker sections.

The temperature changes during early hydration and immediately afterwards can cause large residual thermal stresses as the concrete is hardening. These thermal stresses alone may be large enough to cause transverse deck cracking in some bridges. Reducing the temperature gain during early hydration will reduce thermal stresses and the risk of transverse deck cracking. This can be done by reducing the cement content, using a low heat of hydration cement, cooling the concrete, casting at moderately low ambient temperatures, casting in evening or night conditions, and early wet curing.

The nonuniform temperature gradient that exists when the concrete is hardening represents the stress-free condition. Later, stresses develop even if the section reaches a uniform temperature because of the difference between the stress-free gradient and the later gradient. These residual stresses are similar in nature to those in a rolled-steel section.

The deck of the Portland-Columbia Bridge reached 55°C (130°F) during early hydration, and then cooled to approximately 32°C (90°F) to 13°C (55°F) during the next several days. Daily temperature changes of the deck typically ranged 25°C (45°F). For a temperature decrease of the deck surface of 25°C (45°F), the thermal tensile stresses in the deck are approximately 1.85 MPa (266 psi).

### Diurnal Bridge Temperatures and Stresses

Weather determines the temperatures and thermal stresses in a bridge. The typical diurnal weather cycle begins with the lowest ambient air temperature occurring just before sunrise. Air temperatures then increase as the sun rises, usually peaking a few hours before sunset. After sunset, air temperatures rapidly drop, and the coldest air temperature occurs again just before sunrise. Clouds and precipitation affect this cycle, and often produce the quickest temperature change in a bridge.

Solar radiation is the predominant source of temperature change in most bridges after initial hydration of the cement paste. The deck absorbs part of the radiant energy from the sun, and the remainder is reflected. A dark surface absorbs more radiation energy than a light surface, and a rough surface gains more radiation than a smooth surface. Bridges directly exposed to sunlight will have larger diurnal temperature cycles than shaded bridges in the same geographical region.

Asphaltic concrete overlays are usually much darker in color than portland cement concrete surfaces. An asphaltic concrete overlay absorbs more radiation than a portland cement surface and typically insulates the underlying concrete deck against temperature changes. Consequently, except for overlays thinner than 51 mm (2 in.), asphaltic concrete overlays typically reduce the effects of radiation.

Winds and air currents from moving vehicles affect concrete convection (the transfer of heat from a solid to moving air or fluid). Convection cools deck surfaces, reducing peak temperatures caused by radiation. Convection also accelerates concrete cooling when air temperatures are decreasing. Diurnal bridge temperature cycles are lower when bridges are exposed to faster winds.

For most bridges, diurnal temperature changes produce larger thermal stresses than seasonal temperature changes. Diurnal temperature cycles of concrete decks exposed to solar radiation are often larger than the air temperature cycles, especially when decks are thin. Large girders, especially concrete girders, have large thermal mass and react more slowly to the changing environment; they often have smaller temperature cycles than ambient air has. Bridge decks in moderate or extreme climates can easily experience 28°C (50°F) diurnal temperature cycles.

For some bridges, the greatest temperature changes occur during yearly cycles. As the sun changes position and distance from the earth, the maximum solar radiation incident on the surface occurs on the longest day of the year, and the maximum ambient air temperature typically occurs several days later.

Thermal stresses and the risk of transverse deck cracking are greatest when diurnal temperature cycles are large, solar radiation is high, and large seasonal temperature differences exist. These conditions vary greatly by geographical location, and are unavoidable.

### Seasonal Temperatures and Stresses

Stresses from seasonal temperature change are small or negligible in most concrete bridges because both deck and girders have similar thermal expansion rates. In these bridges, seasonal temperature stresses develop primarily because of different thermal expansion rates of the concrete and reinforcing steel.

When steel girders support a concrete deck, seasonal temperature changes cause stresses when the concrete has a different thermal expansion rate than the steel. Because most concretes have a lower coefficient of thermal expansion than steel has, seasonal temperature decreases will generally cause compressive stresses in the deck, and temperature increases will cause tensile stresses in the deck. A uniform full-depth 28°C (50°F) temperature change in simply supported or continuous-span steel bridges may cause deck stresses as large as 2.0 MPa (291 psi).

## CONCRETE SHRINKAGE AND SHRINKAGE STRESSES

All concrete bridge decks shrink and develop shrinkage stresses. These stresses can be large and cause transverse bridge deck cracking.

### Concrete Shrinkage

Moisture is needed for cement hydration, which stops when the relative humidity (RH) drops to less than approximately 80 percent, for concrete hardening and strength gain. If a high RH is not maintained, concrete dries—primarily by evaporation of the concrete mix water—without experiencing a strength gain.

The major factor affecting the final amount of drying shrinkage is the amount of water in the mix that evaporates. Curing and environment (air temperature, humidity, solar radiation, winds, etc.) affect the shrinkage rate, as does concrete temperature. Poor curing will accelerate drying and shrinkage, accelerate shrinkage stresses, decelerate strength gain, and increase the risk or severity of transverse cracking.

The shrinkage of new concrete occurs in two phases. In the first phase, free water evaporates and a relatively small amount of shrinkage occurs. Then, during the second phase, adsorbed water in capillary and gel pores is lost, and a large amount of shrinkage occurs. Drying shrinkage of new unreinforced, unrestrained concrete in a 23°C (73°F) 50 percent RH environment can range from about 500 to 1000 microstrain ($\mu\epsilon$).

The RH greatly affects drying shrinkage. When concrete reaches temperature and RH equilibrium, its volume remains stable until its humidity or temperature changes. If the humidity decreases, the concrete will shrink, and if the humidity increases, the concrete will swell. Virtually all bridge decks are continuously undergoing RH and temperature cycles because of changing diurnal and seasonal weather conditions.

Concrete surfaces exposed to air dry and shrink quicker than interior locations. After the surfaces have dried, water from interior locations migrates toward the surface and evaporates. Locations near the surface shrink quicker than more interior locations, because the water contained in these areas has a shorter distance to travel to the surface. Theoretically, the shrinkage profile is parabolic, with drying proportional to the square of the distance from the surface.

The evaporation rate of water in concrete depends on the environment and the surface-to-volume ratio of the concrete section. Concretes with a high surface-to-volume ratio dry and shrink quicker (*17*). Members with rapid surface drying and slow diffusion (from either low permeability or large thickness) develop large strain differentials as the surface dries and shrinks while interior portions remain at a high moisture content and shrink much less. Shrinkage differentials initially can produce compressive stresses in the interior and tensile stresses in the exterior portions, contributing to cracking.

Thinner decks shrink quicker and more uniformly than thicker decks. When concrete is cast against a stay-in-place (SIP) form or other moisture barrier that prevents drying from the soffit, shrinkage is slower than when it occurs from both surfaces. Analyses described later in these Guidelines examine both a uniform and a linear shrinkage profile; this simplification was necessary to examine the thousands of material and geometry combinations.

Most drying of a bridge deck will occur within the first 6 to 12 months after construction. During this time shrinkage stresses develop. After concrete dries to a constant moisture level, additional shrinkage is minimal. Decks cast with removable forms will dry faster than decks with SIP forms because they can dry from both top and bottom surfaces. However, shrinkage stresses can be higher when SIP forms are used because drying is less uniform. Reinforced concrete shrinks less than unreinforced concrete because the reinforcement restrains the shrinkage. Reinforcement tends to increase the number of cracks but reduces their individual widths. Aggregates restrain paste shrinkage, and some aggregates restrain paste shrinkage better than others.

Concrete shrinkage is affected by many parameters including aggregate amount and type, amount of mix water, properties and quantities of admixtures, ambient air temperature, RH, curing method, and drying time. Some types of cement cause more shrinkage than others; for example, concrete made with portland cement that is deficient in gypsum will shrink more than a nearly identical concrete with cement that has an optimum gypsum content. ASTM C596 (*15*) can determine the effect of portland cement on the drying shrinkage of a graded Ottawa sand mortar subjected to standard temperature, RH, and evaporation conditions. However, the drying shrinkages of paste or mortar may not be good indicators of the drying shrinkage of a concrete. The following factors produce high shrinkage in concrete:

- High water content,
- High cement content,
- Aggregates with high absorption,
- Aggregates with low moduli of elasticity,
- High air content,
- Aggregates that adhere to entrained air,
- Clay contaminants in aggregates,
- Small maximum aggregate size,
- Lightweight aggregates,
- Calcium chloride admixtures,
- High sand content,
- Triethanolamine admixtures,
- Fine cement,
- Nonoptimum $SO_3$ in cement, and
- Intermediate (3–7 days) length of moist curing.

Shrinkage-compensating cement can reduce concrete shrinkage and shrinkage stresses. The results of using shrinkage-compensating cement to reduce deck cracking are mixed but generally favorable. Additional quality control testing must be done before construction when using shrinkage-compensating cement, including the ASTM C878 (*15*) concrete expansion test. Shrinkage-compensating cements are discussed in detail in the materials section of these Guidelines.

### Shrinkage Stresses

All concrete bridge decks shrink. Shrinkage can cause cracking when the concrete is restrained from shortening. Most bridge girders restrain decks, and such restraint often causes transverse deck cracking. Designers often overlook shrinkage stresses. If bridge girders did not restrain decks, shrinkage would probably not cause deck cracking. The longitudinal stresses that cause transverse deck cracking are caused by a complex interaction of concrete properties, bridge geometry, construction techniques, and geographical environment.

Shrinkage stresses are affected by free-shrinkage, effective modulus of elasticity of the concrete (adjusted for creep), and degree of restraint. Free-shrinkage is the shrinkage of an unrestrained section of concrete, usually measured by prism samples according to AASHTO T160 (*16*). The effective modulus of elasticity is the stress divided by the strain, including creep effects. The degree of restraint is the percentage of free-shrinkage prevented from occurring.

Reinforced concrete shrinks less than unreinforced concrete because the reinforcement restrains the shrinkage. This restraint causes stresses in the deck, but often these stresses are low. In a bridge deck, the girders can largely restrain the deck and cause large stresses and cracking.

Free-shrinkage of the concrete in the new deck of the Portland-Columbia Bridge was approximately 300 $\mu\epsilon$. The restraint varied along the bridge, but typical areas contracted about 130 millionths representing a 43 percent degree of restraint. The concrete modulus of elasticity was approximately 20,000 MPa ($3.0 \times 10^6$ psi) when the concrete was 1 month old. Creep may have reduced the effective modulus of elasticity to 17,200 MPa ($2.5 \times 10^6$

psi). The approximate shrinkage stress in the concrete is the product of the free-shrinkage, the degree of restraint, and the effective modulus, i.e., equal to 300 $\mu\epsilon \times$ 43 percent $\times$ 17,200 MPa ($2.5 \times 10^6$ psi), or 2.22 MPa (322 psi). The tensile strength of the concrete was approximately 2.75 MPa (400 psi). Since the tensile strength of the concrete was greater than the calculated shrinkage stress, cracking in the deck probably did not occur from shrinkage alone. However, thermal stresses can be additive and contribute to the cracking.

## SELF-WEIGHT AND TRAFFIC STRESSES

Traffic loads are rarely a significant cause of transverse deck cracking. In a simply supported span, traffic causes compressive stresses in the deck that do not contribute to transverse cracking. With continuous-span bridges, traffic causes tensile stresses in the deck over supports, and transverse cracking can be worse over these supports. However, transverse cracking often occurs before traffic loads are applied, and it is not limited to the support areas of continuous-span bridges. Although traffic loads may not cause early cracking, traffic vibrations can lead to raveling of early cracks, making them easily visible. Heavier vehicles typically worsen raveling more than lighter vehicles.

CHAPTER 5

# ELASTIC EQUATIONS TO PREDICT SHRINKAGE AND THERMAL STRESSES IN A BRIDGE

## GENERAL DESCRIPTION

Because bridges undergo changing temperatures, and because bridge decks shrink differently than their supporting girders, shrinkage and thermal stresses develop in bridge decks. If a concrete bridge deck was not attached to its supporting girders, shrinkage and temperature changes in the bridge would typically cause the deck and girder to expand and curve differently. However, slippage or separation rarely occurs at the deck-girder interface because of friction and concrete bond, and because of mechanical connections and reinforcement extending across the interface. As such, when shrinkage and temperature changes occur, the girder and deck restrain each other.

A shear force and a force couple (moment) develop at the interface near the ends of the span, (*18*) as shown in Figure 1. When a concrete deck is reinforced, shrinkage and thermal movement of the deck is also resisted by embedded longitudinal reinforcing steel. If present, a SIP metal deck provides additional restraint.

Systems of equations are presented that calculate the forces and force couple in a composite bridge with a reinforced deck, and the resulting stresses. Once these forces and couples are calculated, strains and stresses in the concrete deck can be determined. The derived equations accommodate multiple levels of reinforcement, to include the effects of longitudinal bars and a SIP metal deck. The equations make the following assumptions:

- The concrete deck is fully restrained in the transverse direction, and the girder is unrestrained in the transverse direction.
- Temperature is constant across the width and along the length of the bridge.
- The original temperature of the bridge is uniform. If the original temperature is not uniform, the effects of individual temperature changes can be determined and superimposed on other shrinkage and temperature effects.
- The later temperature of the beam is uniform.
- The later temperature of the deck is either uniform or nonuniform linear with the soffit temperature equal to the beam temperature.
- The reinforcing steel has negligible bending stiffness.
- Separation between the concrete deck and beam does not occur.
- The reinforcing steel in the concrete deck does not slip.

Shrinkage can produce identical stresses to those caused by temperature changes. Shrinkage stresses can be calculated with the derived equations by using equivalent temperature changes. The strain that would develop as a result of the equivalent temperature change and an arbitrary material coefficient of thermal expansion must be the same as the shrinkage for a deck that is perfectly separated from the girders. For example, to calculate the effect of uniform 500 $\mu\epsilon$ free-shrinkage in the deck, the deck's coefficient of thermal expansion may be arbitrarily taken as 1 $\mu\epsilon$/°C (0.56 $\mu\epsilon$/°F) and a 500°C (900°F) temperature decrease is applied to the deck (1 $\mu\epsilon$/°C $\times$ $-500$°C = 0.56 $\mu\epsilon$/°F $\times$ $-900$°F = $-500$ $\mu\epsilon$).

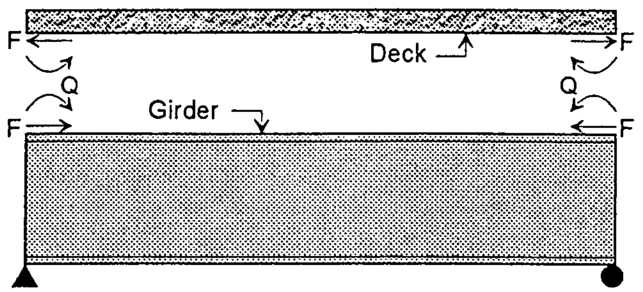


*Figure 1. Compatibility shear force and force couple at beam-girder interface.*

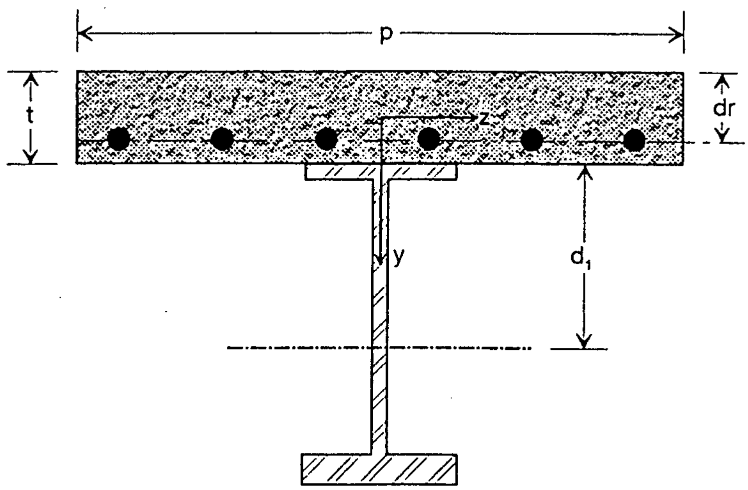


*Figure 2. Bridge cross-section geometry.*

## MATERIAL AND GEOMETRY DEFINITIONS AND ASSUMPTIONS

Figure 2 and the nomenclature listed describe the deck and beam geometry notation used for the derivations.

### NOMENCLATURE

| *Variable* | *Description* |
|---|---|
| $\alpha_{beam}$ | Coefficient of thermal expansion of the beam |
| $\alpha_{deck}$ | Coefficient of thermal expansion of the deck |
| $\alpha_{reinf}$ | Coefficient of thermal expansion of the reinforcement |
| $\epsilon_i$ | Strain in direction i, elongation positive |
| $\mu$ | Poisson's ratio of the deck |
| $\sigma_i$ | Stress in direction i, tensile stresses positive |
| a | Half the deck thickness, = t/2 |
| $A_{beam}$ | Area of the beam |
| $A_{deck}$ | Area of the concrete deck, = pt |
| $d_1$ | Distance to girder centroid from deck soffit |
| $dr_i$ | Depth of deck reinforcement layer i, from upper surface of deck |
| E | Modulus of elasticity |
| $E_{beam}$ | Effective modulus of elasticity of the beam |
| $E_{deck}$ | Effective modulus of elasticity of the deck |
| $E_{reinf}$ | Modulus of elasticity of the deck reinforcement |
| F | Interface shear |
| $Fr_i$ | Force in reinforcement layer i, positive denotes tensile force |
| i | Reinforcement layer number |
| $I_{beam}$ | Moment of inertia of beam |

| | |
|---|---|
| $I_{deck}$ | Moment of inertia of the deck, $= pt^3/12$ |
| nr | Number of reinforcement layers in deck |
| Q | Interface moment (force couple) |
| $S_{deck}$ | Section modulus of deck, $= pt^2/6$ |
| t | Deck thickness |
| $T_0$ | Initial temperature of bridge |
| $T_1$ | Later temperature at upper surface of deck |
| $T_2$ | Later temperature of beam |
| $Tr_i$ | Later temperature of reinforcement layer i |

## SOLUTION OF EQUATIONS

Two sets of equations were derived to analyze strains and stresses in a composite bridge. The first equation set assumes a uniform equivalent-temperature change in the deck and an independent uniform equivalent-temperature change in the girders, as shown in Figure 3. The second assumes the linear temperature change shown in Figure 4.

When the concrete deck is separated from the beam, it will be acted upon by a shear force at the interface with the beam (F), and a force couple (Q) near each end of the span. The concrete is also acted upon by the forces in the reinforcement ($Fr_{i..nr}$). For *nr* layers of reinforcement, there are 2 + *nr* variables that must be solved for: F, Q, and $Fr_{i..nr}$.

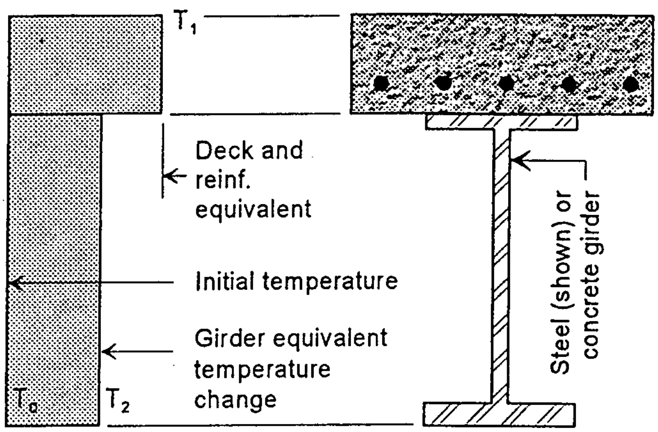


*Figure 3. Condition 1, Uniform temperature change in concrete deck and uniform temperature change in girder.*

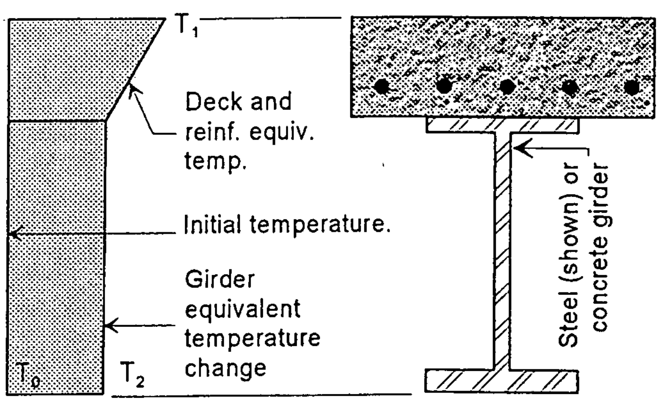


*Figure 4. Condition 2, Linear temperature change in concrete deck and uniform temperature change in girder.*

The unknown forces (F and $Fr_{1..nr}$) and force couple (Q) can be determined by solving a number of equations. An exact symbolic solution contains thousands of terms and is impractical. It is recommended that users first reduce the number of variables by substituting real values for known variables, then solve the reduced system either symbolically or numerically. Several low-cost mathematical software packages are currently available that have sufficient power to solve for the unknown forces and force couple when the other variables have been defined.

Equations 1, 2, and 3 must be solved to calculate the restraint when a bridge is subjected to a uniform temperature (or equivalent temperature) change. Note that Equation 5 must be solved once for each layer (i) of reinforcement.

$$\varepsilon_{deckBot} = \frac{2(1-\mu^2)}{E_{deck}pt^2}\left[-3\left(Q + \sum_{i=1}^{nr}(dr_iFr_i)\right) + t\left(2F + \sum_{i=1}^{nr}(Fr_i)\right)\right]$$
$$+\alpha_{deck}(1+\mu)(T_1 - T_0) = \frac{-1}{E_{beam}}\left(\frac{F}{A_{beam}} + \frac{d_1^2F}{I_{beam}} + \frac{d_1Q}{I_{beam}}\right) + \alpha_{beam}(T_2 - T_0) \qquad \text{Eq. 1}$$

$$\frac{6(1-\mu^2)\left[-2\left(Q + \sum_{i=1}^{nr}(dr_iFr_i)\right) + t\left(F + \sum_{i=1}^{nr}Fr_i\right)\right]}{E_{deck}pt^3} = \frac{d_1F + Q}{E_{beam}I_{beam}} \qquad \text{Eq. 2}$$

$$\frac{1-\mu^2}{E_{deck}p}\left[\left(\frac{6}{t^2} - \frac{12dr_i}{t^3}\right)\left(Q + \sum_{i=1}^{nr}(Fr_idr_i)\right) + \left(\sum_{i=1}^{nr}Fr_i\right)\left(\frac{6dr_i}{t^2} - \frac{4}{t}\right) + F\left(\frac{6dr_i}{t^2} - \frac{2}{t}\right)\right]$$
$$+\alpha_{deck}(1+\mu)(T_1 - T_0) = \frac{Fr_i}{Ar_iE_{reinf}} + \alpha_{reinf}(Tr_i - T_0) \qquad \text{Eq. 3}$$

Equations 4, 5, and 6 must be solved to calculate the restraint when a bridge is subjected to a linear temperature (or equivalent temperature) change. Note that Equation 8 must be solved once for each layer of reinforcement.

$$\frac{2(1-\mu^2)}{E_{deck}pt^2}\left[-3\left(Q + \sum_{i=1}^{nr}(dr_iFr_i)\right) + t\left(2F + \sum_{i=1}^{nr}Fr_i\right)\right] + \alpha_{deck}(1+\mu)(T_2 - T_0)$$
$$= \frac{-1}{E_{beam}}\left(\frac{F}{A_{beam}} + \frac{d_1^2F}{I_{beam}} + \frac{d_1Q}{I_{beam}}\right) + \alpha_{beam}(T_2 - T_0) \qquad \text{Eq. 4}$$

$$\frac{1-\mu^2}{E_{deck}p}\left[\frac{-12}{t^3}\left(Q + \sum_{i=1}^{nr}(Fr_idr_i)\right) + \frac{6}{t^2}\left(F + \sum_{i=1}^{nr}Fr_i\right)\right]$$
$$\frac{+\alpha_{deck}(1+\mu)(T_2 - T_1)}{t} = \frac{d_1F + Q}{E_{beam}I_{beam}} \qquad \text{Eq. 5}$$

$$\frac{1-\mu^2}{E_{deck}p}\left[(Q+\sum_{i=1}^{nr}(Fr_i dr_i))\left(\frac{6}{t^2}\ \frac{12dr_i}{t^3}\right)+F\left(\frac{6dr_i}{t^2}-\frac{4}{t}\right)\sum_{i=1}^{nr}Fr_i\right]$$

$$+\alpha_{deck}(1+\mu)\left[\frac{dr_i}{t}(T_2-T_1)+T_1-T_0\right]=\frac{F_{rl}}{A_{rl}E_{reinf}}+\alpha_{reinf}(Tr_i-T_0) \qquad \text{Eq. 6}$$

After the girder and reinforcement restraints are calculated, the stress at the bottom and top surfaces of the deck can be determined from Equations 7 and 8, respectively.

$$\sigma_{deckBot}=\frac{F-\sum_{i=1}^{nr}Fr_i}{A_{deck}}+\frac{Fa-Q+\sum_{i=1}^{nr}[Fr_i(a-dr_i)]}{S_{deck}} \qquad \text{Eq. 7}$$

$$\sigma_{deckTop}=\frac{F-\sum_{i=1}^{nr}Fr_i}{A_{deck}}-\frac{Fa-Q+\sum_{i=1}^{nr}[Fr_i(a-dr_i)]}{S_{deck}} \qquad \text{Eq. 8}$$

CHAPTER 6

# ANALYTICAL PARAMETER STUDY

## BACKGROUND

Transverse cracks develop in a concrete bridge deck when longitudinal tensile stresses in the deck exceed the tensile strength of the concrete. The analytical studies used conventional and finite element analysis techniques to evaluate the influence of various parameters on deck tensile stresses and cracking. These factors included concrete drying shrinkage, creep, hydration temperatures and other thermal effects, position and amount of reinforcing steel, girder size and spacing, single- and multispan conditions, and other parameters. The combined effects of material properties and geometry were analyzed. These analytical techniques enabled a more thorough investigation of the parameters that affect cracking than field work or laboratory testing alone could have.

To perform the analytical parameter study, equations were derived to calculate stresses for specific shrinkage or temperature changes for any combination of geometry and material properties. These equations assume an ideal uniform or linear shrinkage or temperature change in the deck. The study examined more than 18,000 combinations of materials and geometry. This project examined elastic stresses in composite bridges, for a wide variety of bridge geometries and material properties, subjected to constant and linear deck shrinkage and temperature changes. This work determined which material properties and geometries develop larger shrinkage and thermal stresses and are more likely to cause early deck cracking.

Before these elastic equations were used and the parameter study started, the equations were used to predict theoretical thermal and shrinkage behavior of the Portland-Columbia Bridge. The calculated theoretical strains compared well to actual measured strains for various temperature changes and shrinkage, indicating that theory can approximate real behavior. Although elastic analysis cannot predict stresses after cracking, it indicates if cracking will occur, with higher elastic stresses predicting more extensive cracking.

The ideal uniform and linear shrinkage and temperature profiles are simplified approximations of profiles that often actually occur. The Portland-Columbia Bridge developed significant nonlinear temperatures that cannot be approximated by the simplified uniform or linear temperature profile used for the analyses of geometry and materials. All outdoor bridges also develop nonlinear temperature gradients. However, examination of measured reinforcement strains in the Portland-Columbia Bridge shows that extreme or unusual strains and stresses did not occur in the deck reinforcement from these nonlinear temperature changes, and hence probably not in the deck. Nonlinear temperature changes can produce larger stresses than an ideal uniform or linear temperature change, but the ideal temperature changes can approximate representative thermal stresses.

## STRESSES IN SIMPLY SUPPORTED VERSUS CONTINUOUS-SPAN BRIDGES

Shrinkage and thermal stresses in simply supported spans are generally uniform along the length of the span; stresses are different within a few feet of the ends of the spans, where interface forces develop. Shrinkage and the stresses in a simply supported bridge are unaffected by span length. Nonuniform shrinkage and temperature strain in the deck and girders cause curvature but do not affect support reactions in a simply supported span.

However, with continuous-span bridges, these curvature strains alter the support reactions and, therefore, the stresses within the deck. For example, consider two bridges each with the same total length, identical cross-section geometry, and identical material properties; the only difference is that one bridge is simply supported, and the other is continuous across two spans. If the top surface of the simply supported bridge is heated relative to the girders, convex-upward curvature occurs and the center of the span deflects upward. The two-span bridge reacts in the same way, except the interior support pulls the bridge downward so that there is no vertical displacement at its location (Figure 5). This causes additional stresses to develop in the bridge, stresses that vary linearly between supports. The vertical force at an interior support affects shrinkage and thermal stresses the same way a vehicle at the same location affects gravity-load stresses in a simply supported bridge.

For a given cross-section, shrinkage and thermal stresses at an interior support are affected by span-length proportions and not actual span lengths. For another example, consider a two-span bridge with identical materials and cross-section, with an interior support at mid-length; shrinkage and thermal stresses above the interior support are the same regardless if the actual span lengths are 50 m (164 ft) or 80 m (262 ft), provided that the interior support is at mid-length. Similarly, stresses above the interior support of a continuous two-span bridge with span lengths of 20 m (66 ft) and 40 m (131 ft) are the same as a bridge with span lengths of 60 m (197 ft) and 120 m (394 ft), because the span proportion is constant at 1:2. The effects of continuity are largest in a two-span bridge with equal span lengths.

## THERMAL STRESSES

Large tensile stresses can develop in the deck after peak hydration temperatures are reached and the deck cools. For example, a 28°C (50°F) temperature drop in the deck relative to the girders can cause elastic tensile stresses greater than 1380 kPa (200 psi) when the concrete deck has an early modulus of elasticity of only 3450 MPa ($0.5 \times 10^6$ psi), and greater than 6900 kPa (1000 psi) when the modulus is a more mature 17,200 MPa ($2.5 \times 10^6$ psi).

The upper surface of the deck typically heats and cools quicker, because it is exposed to direct solar radiation and precipitation, and temperature gradients usually exist. A linear temperature gradient in the deck, not a uniform temperature gradient, typically produces larger deck stresses. For example, a temperature increase in the deck that linearly varies from 28°C (50°F) at its top surface to 0°C (0°) at its soffit will cause larger stresses than a uniform 28°C (50°F) temperature increase in the deck.

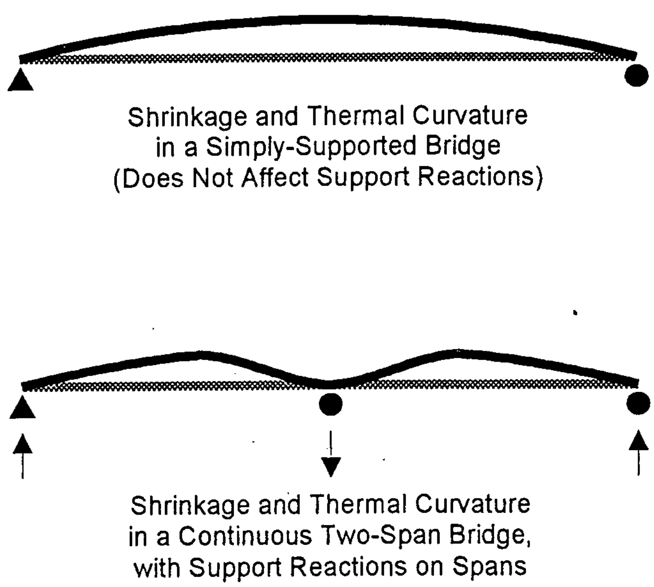


*Figure 5.* *Continuous two-span structure.*

In a simply supported bridge, a 28°C (50°F) diurnal temperature cycle in the deck may cause stresses in the deck as large as 9.3 MPa (1350 psi) with steel girders, and 10.2 MPa (1480 psi) with concrete girders. When spans are continuous over supports, the same temperature cycle can cause stresses as large as 13.8 MPa (2000 psi) with concrete girders; continuity typically does not significantly affect deck stresses in steel-girder bridges.

## SHRINKAGE STRESSES

Shrinkage is never uniform because drying is never uniform. Drying shrinkage causes stresses to develop in the deck. The shrinkage stresses alone may be large enough to cause transverse deck cracking, or, when combined with thermal stresses, they may cause transverse cracking.

Shrinkage stresses are generally worse in a steel-girder bridge than in a concrete-girder bridge. This is usually the case because the steel girders do not shrink with the concrete deck, and the shrinkage strain difference between the deck and girders is maximized. Shrinkage stresses are often lowest in a monolithic cast-in-place bridge, where both the deck and girders have similar final shrinkages (although the deck typically shrinks quicker than the thicker girders). A bridge deck supported by precast, prestressed girders may develop more or fewer shrinkage stresses than the monolithic concrete bridge, depending on the remaining shrinkage and creep of the precast girders when the concrete deck is cast.

When the deck shrinks relative to its girders in a simply supported bridge, 500 $\mu\epsilon$ uniform free-shrinkage of the concrete deck may cause tensile stresses as large as 9650 kPa (1400 psi) in a steel-girder bridge and 12,400 kPa (1800 psi) in a concrete-girder bridge, depending on geometry and material properties; maximum stresses from a linear shrinkage profile are slightly larger.

For some steel-girder bridges, shrinkage stresses are largest over interior supports, but for the other bridges, the stresses are lower over the supports; continuity effects are generally small with steel-girder bridges. Stresses above the interior supports of a monolithic (uniformly shrinking) concrete bridge are not significantly affected by continuity over the support. However, when differential shrinkage occurs between the concrete girders and the deck, total stresses over the interior support may reach nearly 13,800 kPa (2000 psi) for differential shrinkage of 500 $\mu\epsilon$, much larger than the stresses away from support.

## PARAMETER STUDY FINDINGS

Shrinkage and temperature change can cause large tensile stresses in bridge decks. The parameter study determined the stresses caused by different levels of shrinkage and temperature changes in decks of simply supported and continuous-span bridges with steel and concrete girders.

The calculated stresses show that thermal and shrinkage stresses in a concrete bridge deck can be several times larger than the tensile strength of concrete. While tensile stresses in concrete can never be as high as these calculated stresses, the calculated stresses indicate that cracking is expected. Higher calculated stresses predict more severe cracking.

While theory predicts that many combinations of geometry and material properties will cause cracking, it predicts that others will not. Usually, but not always, the largest stresses were calculated with the largest girders at the narrowest spacing, the thinnest decks, and the highest concrete modulus of elasticity. Designers should investigate the shrinkage and thermal stresses that can develop in a bridge deck. Designs that are expected to have high stresses that may cause stresses should be avoided when possible, or additional reinforcement should be added to the deck to control the expected cracking.

# CHAPTER 7
# DESIGN

## INTRODUCTION

The parameter study found that bridge geometry can have a large effect on deck stresses and transverse cracking. This chapter summarizes the results of the parameter study related to design, supplemented with knowledge gained from the survey response and literature review. It also presents design recommendations to reduce deck stresses and the risk of transverse cracking.

Most states build similar bridges. Tables 4 and 4(a) show the bridge geometries currently being built in the United States.

Current AASHTO design procedure requires that bridge design account for longitudinal movement at the supports resulting from temperature changes but does not require examination of shrinkage (except with prestressed bridges) and nonuniform temperatures. The AASHTO procedure is adequate to calculate movement that must be accommodated at supports, but it does not address stresses that occur from shrinkage and daily temperature changes.

Shrinkage and temperature changes are the primary cause of transverse deck cracking. Bridge designers rarely consider these stresses, because shrinkage and temperature-reinforcing steel are usually considered sufficient to control cracking. Designers should consider these stresses in their designs, using the equations presented in the preceding chapter or other methods.

## SPAN SUPPORT

When spans are simply supported, shrinkage and thermal stresses are generally uniform along the length of the bridge. The reactions at the supports are not affected by these stresses if the bearings can move to accommodate the associated longitudinal movement. The supports do not restrain the bridge curvature caused by shrinkage and temperature changes.

However, when spans are continuous over supports, the interior supports restrain the curvatures caused by shrinkage and temperature changes. This external restraint applies additional stresses to the deck. These additional stresses generally vary linearly between supports. Some transportation agencies found more transverse cracking on continuous-span bridges than on simply supported bridges. Researchers (*19*) found more cracking in the middle spans of continuous structures than in the end spans, and cracking was slightly more prevalent on two-span systems than on other continuous-span systems.

The parameter study found that continuity over supports usually affected bridges with concrete girders more than those with steel girders. However, the effect depends on the curvature of the composite bridge caused by shrinkage and temperature changes, and the section modulus of the composite section. The effect on support reactions is proportional to the moment of inertia of the composite bridge, and the curvature that would occur without the interior supports. The associated stress at the upper surface of the deck is then proportional to this reaction effect, and inversely proportional to the composite section modulus.

TABLE 4 Typical geometry of bridge decks based on the survey results (Metric) (English conversion in Table 4{a})

| Continuous spans—typical range | Cast-in-place reinforced concrete girder | Post-tensioned concrete girder | Precast, prestressed concrete girder | Steel girder |
|---|---|---|---|---|
| Span lengths (m) | 6.1-60.9 / 16.7-33.5 | 21.3-195 / 33.5-85.3 | 7.6-94.5 / 13.7-38.1 | 6.1-160 / 27.4-76.2 |
| Girder spacings (m) | 1.2-3.0 / 1.7-2.6 | 2.0-3.7 / 2.4-3.0 | 1.2-4.3 / 2.0-3.0 | 1.2-6.1 / 2.1-3.4 |
| Girder depth (m) | 0.3-2.4 / 0.9-1.4 | 0.9-3.7 / 1.5-2.9 | 0.3-2.4 / 0.8-1.7 | 0.6-0.9 / 0.8 |
| Top steel cover (mm) | 38-75 / 50 | 38-75 / 63 | 38-75 / 63 | 50-75 / 63 |
| Bottom steel cover (mm) | 25-38 / 25 | 25-38 / 25 | 25-38 / 25 | 25-50 / 25 |
| Slab thickness (mm) | 165-304 / 177 | 177-250 / 190-228 | 152-279 / 190-241 | 177-381 / 203-250 |
| Maximum bar size in deck (mm) | 19-35 / 19 | 12-35 / 22 | 16-35 / 19 | 12-35 / 19 |
| **Simple spans—typical range** | | | | |
| Span lengths (m) | 6.1-38 / 12.1-21.3 | 22.8-67.0 / 30.4-53.3 | 6.1-45.7 / 13.8-36.5 | 6.1-76.2 / 21.3-48.8 |
| Girder spacings (m) | 0.9-3.0 / 1.5-2.4 | 2.0-3.7 / 2.1-3.0 | 1.2-4.8 / 1.8-3.0 | 1.2-4.2 / 2.1-3.0 |
| Girder depth (m) | 0.3-2.4 / 0.8-1.5 | 1.2-3.0 / 2.3-3.0 | 0.1-2.4 / 0.8-1.8 | 0.3-3.7 / 0.7-2.1 |
| Top steel cover (mm) | 38-63 / 50 | 38-75 / 50 | 38-75 / 57 | 38-75 / 63 |
| Bottom steel cover (mm) | 25-50 / 8.5 | 25-38 / 25 | 25-50 / 25 | 25-38 / 25 |
| Slab thickness (mm) | 152-250 / 288 | 101-457 / 177-279 | 152-292 / 190-228 | 102-279 / 152 |
| Maximum bar size in deck (mm) | 12-35 / 22 | 16-35 / 22-25 | 12-35 / 19 | 12-35 / 19 |
| **General** | | | | |
| Typical spacing of temperature reinforcing steel (mm) | 304-457 / 304 | 304-457 / 304 | 152-457 / 304 | 304-457 / 304 |
| Which bars are nearest the top surface of the slabs?<br>Transverse-longitudinal-varies with design<br>(T L V) [number of responses] | T - 9<br>L - 0<br>V - 2 | T - 10<br>L - 2<br>V - 0 | T - 29<br>L - 5<br>V - 0 | T - 34<br>L - 3<br>V - 1 |
| Are top and bottom transverse bars aligned?<br>(Yes No) [number of responses] | Y - 9<br>N - 1 | Y - 8<br>N - 3 | Y - 25<br>N - 8 | Y - 30<br>N - 8 |

Note: First set of values specify range of values obtained from the agencies. The second set of values are typical ranges.

TABLE 4(a) Typical geometry of bridge decks based on the survey results

| **Continuous spans—typical range** | **Cast-in-place reinforced concrete girder** | **Post-tensioned concrete girder** | **Precast, prestressed concrete girder** | **Steel girder** |
|---|---|---|---|---|
| Span lengths (ft) | 20-200 / 55-110 | 70-640 / 110-280 | 25-310 / 45-125 | 20-525 / 90-250 |
| Girder spacings (ft) | 4-10 / 5.5-8.5 | 6.5-12 / 8-10 | 4-14 / 6.5-10 | 4-20 / 7-11 |
| Girder depth (ft) | 1-8 / 3-4.5 | 3-12 / 5-9.5 | 1-8 / 2.5-5.5 | 2-12 / 3.5-9 |
| Top steel cover (in.) | 1.5-3 / 2 | 1.5-3 / 2.5 | 1.5-3 / 2.5 | 2-3 / 2.5 |
| Bottom steel cover (in.) | 1-1.5 / 1 | 1-1.5 / 1 | 1-1.5 / 1 | 1-2 / 1 |
| Slab thickness (in.) | 6.5-12 / 7-8.5 | 7-10 / 7.5-9 | 6-11 / 7.5-9.5 | 7-15 / 8-10 |
| Maximum bar size in deck (No.) | 6-11 / 6 | 4-11 / 7 | 5-11 / 6 | 4-11 / 6 |
| **Simple spans—typical range** | | | | |
| Span lengths (ft) | 20-125 / 40-70 | 75-220 / 100-175 | 20-150 / 45-120 | 20-250 / 70-160 |
| Girder spacings (ft) | 3-10 / 5-8 | 6.5-12 / 7-10 | 4-16 / 6-10 | 4-14 / 7-10 |
| Girder depth (ft) | 1-8 / 2.5-5 | 4-10 / 7.5-10 | 0.5-8 / 2.5-6 | 1-12 / 3-7 |
| Top steel cover (in.) | 1.5-2.5 / 2 | 1.5-3 / 2 | 1.5-3 / 2.25 | 1.5-3 / 2.5 |
| Bottom steel cover (in.) | 1-2 / 1 | 1-1.5 / 1 | 1-2 / 1 | 1-1.5 / 1 |
| Slab thickness (in.) | 6-10 / 9 | 4-18 / 7-11 | 6-11.5 / 7.5-9 | 7-11 / 8-9.5 |
| Maximum bar size in deck (No.) | 4-11 / 7 | 5-11 / 7-8 | 4-11 / 6 | 4-11 / 6 |
| **General** | | | | |
| Typical spacing of temperature reinforcing steel (in.) | 12-18 / 12 | 12-18 / 12 | 6-18 / 12 | 12-18 / 12 |
| Which bars are nearest the top surface of the slabs?<br>Transverse-longitudinal-varies with design<br>(T L V) [number of responses] | T - 9<br>L - 0<br>V - 2 | T - 10<br>L - 2<br>V - 0 | T - 29<br>L - 5<br>V - 0 | T - 34<br>L - 3<br>V - 1 |
| Are top and bottom transverse bars aligned?<br>(Yes No) [number of responses] | Y - 9<br>N - 1 | Y - 8<br>N - 3 | Y - 25<br>N - 8 | Y - 30<br>N - 8 |

**Note:** First set of values specify range of values obtained from the agencies. The second set of values are typical ranges.

## GIRDER DESIGN

The concrete deck of a composite bridge is externally restrained by its supporting girders. Transverse deck cracking would not occur without this girder restraint. However, noncomposite bridges are usually not economically practical. Even when a bridge is designed to be noncomposite, some composite behavior is inevitable from simple friction at the girder-deck interface. For most bridges, the girder affects deck stresses and transverse cracking more than any other design factor.

### Girder Type

Some transportation agencies found more transverse cracking in decks supported by steel girders than those supported by concrete girders (*19,20*). One researcher (*21*) found more cracking for steel girders when SIP forms were used but much more cracking for precast girders when conventional forms were used.

The analytical studies found that stresses and the risk of transverse deck cracking are usually higher when decks are supported by steel girders instead of concrete girders. Steel is more thermally conductive than concrete, and larger temperature variations and thermal stresses can occur with steel girders. Also, steel girders do not shrink but concrete girders do, so the differential shrinkage of the deck and girder is usually much higher with steel girders, and the risk or severity of transverse cracking is greater. However, with continuous spans, the risk or severity of localized transverse deck cracking over interior supports may be worse with concrete girders.

### Girder Size and Spacing

Larger girders restrain bridge decks more than smaller girders, and the risk of transverse cracking can be expected to be worse with larger girders. Because longer spans require larger girders, longer spans may be more prone to transverse deck cracking. Reducing the girder spacing typically increases deck stresses for a given girder size, but reducing the spacing also reduces the required girder size, and the effects are generally offsetting.

Girder size and spacing are generally dictated by the bridge span. When practical, reducing the span and girder size can decrease deck stresses and the risk of transverse cracking. Often bridges with longer spans crack more than those with shorter spans (*19,21*), and spans longer than 21 m to 30 m (70 to 90 ft) may be most susceptible to cracking (*19*).

### Dead-Load Deflection

Some states attributed transverse deck cracking to excessive dead-load deflections, but other researchers found no such relationship (*21*). Transverse cracking of the plastic concrete can occur over the supports of unshored continuous-span bridges as a result of dead-load bending. One researcher found concrete placed in flexible formwork simulating deck slabs cracked between 2 and 4.5 hrs after mixing, when a curvature was of 0.013 $mm^{-1}$ ($5x10^{-4}$ $in.^{-1}$) (*22*). To prevent this cracking, falsework deflections should be calculated and placement sequences selected to limit negative moments in continuous-spans bridges. However for most bridges, dead-load deflection is not a primary cause of transverse deck cracking.

## DECK DESIGN

The design of the deck usually has a moderate or small effect on deck stresses. The longitudinal stresses that can cause transverse deck cracking are affected by the deck's thickness; the cover over its reinforcement; the reinforcement amount, size, spacing, and alignment; the concrete properties; embedded studs; form type; and post-tensioning, if any. The following subsections discuss the aspects of deck design that can affect transverse bridge deck cracking.

### Deck Thickness

The parameter study found that thicker decks usually develop smaller shrinkage and thermal stresses for a specific shrinkage pattern or temperature change, but the effect is sometimes inconsistent. Uniform shrinkage or temperature changes in the deck usually develop nearly uniform stresses in thin decks; whereas thicker decks on smaller girders may develop significant bending stresses. Thicker decks are also more prone to develop nonuniform shrinkage and temperatures than thinner decks, which may increase stresses.

Recent research suggests that bridge decks thinner than 230 to 254 mm (9 to 10 in.) are more susceptible to cracking than thicker decks (*19,23*). One highway department (*24*) found that thickening bridge decks from 162 mm (6.4 in.) to 218 mm (8.6 in.) reduced deck cracking, especially those cracks wider than 0.13 mm (0.005 in.). However, this department concluded that the cost of thicker decks outweighed the benefit and does not recommend increasing deck thickness to reduce cracking.

### Concrete Cover

The analytical studies found that reinforcement depth has an inconsistent effect on deck stresses. Two field studies found that concrete decks with more than 75 mm (3 in.) of cover had more transverse deck cracking than decks with less cover (*19,25*). This contradicts others that found no correlation between cracking and cover (*26,27*). Because the probability of settlement cracking decreases as the clear cover increases, decks with low cover are more prone to settlement cracking. However, moving reinforcing bars farther from the concrete surface reduces their effectiveness in distributing shrinkage stresses and reducing crack widths at the surface.

Cover from 38 to 75 mm (1.5 to 3 in.) is generally recommended (*19*). A minimum cover of 50 mm (2 in.) is often necessary to avoid settlement cracking and is recommended for corrosion protection of decks subjected to deicing chemicals.

### Reinforcing Bar Size

Reducing the bar size (and decreasing bar spacing to maintain the same reinforcement ratio) will reduce stress concentration and crack widths. *NCHRP Report 297* (*28*) recommended small reinforcing bars to reduce cracking, but did not provide specific values. Smaller bars also reduce the probability of settlement cracking. One researcher (*23*) recommended reducing the maximum bar size to 16 mm (⅝ in.).

### Reinforcing Bar Type

Bridge decks subjected to deicing chemicals or sea spray should be reinforced with epoxy-coated reinforcement, stainless steel, or other corrosion-resistant reinforcement. These reinforcements resist corrosion and thereby reduce deck maintenance.

Researchers (*29,30*) found that epoxy-coated reinforcement typically causes wider but fewer cracks in reinforced beams tested in flexure. However, one highway department found that bridge decks reinforced with epoxy-coated bars cracked more than those with plain steel bars (*19*). Because the concrete bond and slip strength is less when bars are epoxy-coated, beams with epoxy-coated reinforcement often develop wider cracks. However, epoxy-coated bars resist corrosion better, and will improve long-term deck performance despite wider cracks.

Stainless steel, corrosion-resistant alloy steel, or reinforcing bars having improved organic coatings may best resist corrosion in very aggressive continuously wet environments. Although stainless steel and other types of corrosion-resistant bars are more expensive than epoxy-coated reinforcement, they reduce long-term maintenance costs and may provide the most cost-effective long-term reinforcement in corrosive environments. Using a highly corrosion-resistant steel may address corrosion concerns but not the effects of leakage through the deck if cracking occurs.

### Bar Alignment

Most transverse deck cracks develop directly above the top reinforcing steel bars (*20,31*). When the top and bottom transverse bars align, they form a weakened section within the concrete, which may be more susceptible to cracking. Full-depth cracking usually occurs through both top and bottom transverse bars when the bars align. By offsetting the top and bottom transverse bars, the risk or severity of transverse cracking may be slightly reduced.

### Quantity of Reinforcement

The amount and location of longitudinal deck reinforcement typically have a small effect on stresses in the concrete deck of a steel-girder bridge. More reinforcement causes larger deck stresses, but the additional stresses are usually small or negligible. Increasing the amount of longitudinal reinforcement in the deck may slightly increase the risk of transverse cracking.

However, longitudinal reinforcement controls transverse cracking, and affects transverse crack widths and deck serviceability. The minimum longitudinal reinforcement required by the AASHTO specifications has been insufficient to control cracking. The requirement should be increased to reduce crack widths and improve deck serviceability. As a minimum, longitudinal size 10M (slightly smaller than a #4 bar) bars should be placed at a maximum spacing of 150 mm (6 in.).

## CONCRETE STRENGTH

High-strength concretes generally crack more than lower-strength concretes, primarily because high-strength concretes typically contain more cement and may have more shrinkage and higher heats of hydration. Also, high-strength concrete usually has a high modulus of elasticity and low creep, and therefore develops high shrinkage stresses for a given restrained shrinkage or thermal strain. To reduce transverse cracking, designers should not specify high-strength concrete for bridge decks. Instead, designers should specify *maximum* cement contents or strengths to reduce shrinkage and thermal cracking. Durable, low-permeability concretes can still be achieved with lower cement contents and strengths.

## STUD SPACING

The steel studs or channels commonly used to provide shear transfer at the girder-deck interface of composite beams cause local stress concentrations. Very little information is published on the effect of these connections on concrete cracking. Arkansas DOT tried but could not correlate cracking to stress concentrations at these connectors. Structural finite element analyses for this project calculated concrete stress concentrations in areas surrounding steel studs to be approximately 20 percent higher than the average stress in the deck.

## POST-TENSIONED DESIGN

AASHTO design procedures permit tensile stresses in longitudinally post-tensioned decks and do not require additional reinforcement if the stresses do not exceed a certain limit; for large tensile stresses, reinforcement in these tensile zones is required to control (not prevent) cracking. For many design requirements, designing the post-tensioning to produce tensile stresses will result in a more efficient design and, therefore, these designs indirectly encourage tensile stresses. The risk of transverse cracking is increased, especially when additional reinforcement is not placed in tensile zones created by the longitudinal post-tensioning. Designing the longitudinal post-tensioning to produce compressive stresses in the deck will generally decrease the risk of severity of transverse deck cracking, but decrease the flexural efficiency of the system.

### FORM TYPE

There is debate about whether SIP forms or removable forms cause more cracking. Some researchers found that SIP forms reduced transverse cracking (*21*), while others did not (*32*). The SIP forms prevent drying from the soffit, reducing the average shrinkage of the deck; however, this project's parameter study found that nonuniform shrinkage can often cause larger tensile stresses at the upper deck surface and increase the risk or severity of cracking. These forms also hide soffit cracks from view and prevent inspection of the underside. Steel SIP forms can hold moisture against the concrete and corrode when water leaks through deck cracks.

### SKEW

Skew does not significantly affect transverse cracking, except that slightly higher stresses occur near the corners. One researcher (*23*) found bridges with skews greater than 30 degrees to be more susceptible to transverse cracking.

### TRAFFIC VOLUME

Researchers (*33*) found that bridges with high traffic volume tended to have more cracks than those subjected to lower volumes, although the trend was not clearly shown. Heavy truck traffic appeared to extend the length of cracks, but did not significantly affect leakage. Other researchers found that average daily traffic did not affect deck cracking (*21*,*34*). High traffic volumes may ravel cracks and make them more visible.

### FREQUENCY OF TRAFFIC-INDUCED VIBRATION

Traffic vibrations can cause very small tensile stresses in bridge decks. One researcher (*35*) determined that traffic-induced vibrations do not adversely affect concrete decks. Others also found that vibration frequency does not affect transverse cracking (*36*). Investigations of bridge deck widening did not show any problems as a result of traffic-induced vibrations.

### ALLOWABLE STRESS VERSUS ULTIMATE STRENGTH DESIGN

Some researchers believe that changing the design method from allowable stress to ultimate strength has produced more flexible structures that are more susceptible to cracking (*23*,*28*). However, one researcher found that flexibility does not affect cracking (*21*).

CHAPTER 8

# CONCRETE PROPERTIES AND MIX DESIGN

## GENERAL

The parameter study and the survey responses indicate that concrete material properties play a primary role in transverse deck cracking. Transportation agencies should design and evaluate concrete mixes to reduce or eliminate transverse deck cracking. Trial batches and mix parameters can be tested with the restrained ring test.

The most important concrete properties affecting deck stresses and cracking are the concrete modulus of elasticity, creep, shrinkage, and the coefficient of thermal expansion. Shrinkage strains and stresses are directly proportional to the free-shrinkage of the concrete, and thermal strains and stresses are largely affected by the concrete coefficient of thermal expansion. For a given restraint, the shrinkage and thermal strains are largely influenced by the concrete's modulus of elasticity and its creep. The aggregate type and volume, cement content, and cement type largely affect the concrete modulus of elasticity, creep, shrinkage, and the coefficient of thermal expansion.

Mixes having the lowest cracking tendency should be selected for use in bridge decks. The following material effects are provided to help in the development of low-cracking-tendency mixes; however, testing of actual mixes is recommended.

## CONCRETE MODULUS OF ELASTICITY AND CREEP

The concrete modulus of elasticity and especially creep influence thermal and shrinkage stresses more than any other material properties, despite girder type or bridge geometry. In fact, because girder stiffness (the primary deck restraint) is primarily dictated by span length, the concrete modulus of elasticity and creep are the two most important factors that can be controlled to reduce transverse deck cracking.

The combination of the modulus of elasticity and creep determines the stress that develops as a result of the shrinkage and thermal strains. Reducing the concrete modulus of elasticity or increasing creep, or both, reduces both shrinkage and thermal stresses, and reduces the risk of transverse deck cracking.

Diurnal temperature changes and corresponding thermal stresses change quickly, and creep does not significantly affect diurnal thermal stresses. On the other hand, shrinkage occurs over a long time and creep can significantly reduce shrinkage stresses. Selecting a concrete with a low modulus of elasticity and high creep generally will reduce shrinkage and thermal stresses, and reduce the risk or severity of transverse deck cracking.

The simplest way to reduce the concrete modulus of elasticity is to use lower-strength concrete. Creep can be increased by reducing the concrete strength, increasing the paste content, and selecting aggregates with low moduli of elasticity. The porosity and absorption of the aggregate may be related to creep because of its role in the transfer of moisture within the concrete. Other design factors also affect the concrete modulus of elasticity and creep, as discussed in the following sections.

## CONCRETE STRENGTH

As the compressive strength of concrete increases, generally both the modulus of elasticity and the tensile strength also increase. As such, the tensile stresses that cause cracking and the strength of the concrete to resist cracking both increase. Standard American Concrete Institute (ACI) equations stipulate that both tensile strength and modulus of elasticity be proportional to the square-root of the compressive strength. Because the ability to resist cracking is linearly proportional to the tensile strength, the standard ACI equations predict that the risk of transverse deck cracking decreases as the concrete compressive strength increases.

Creep is the concrete property that has the largest effect on long-term deck stresses and transverse deck cracking. Creep reduces stresses that may lead to cracking. Generally, as concrete compressive strength increases, creep decreases much quicker than the modulus of elasticity and the tensile strength increase. In other words, increasing the concrete compressive strength usually increases restrained tensile stresses more than the tensile strength of the concrete increases, resulting in a higher risk of transverse bridge deck cracking.

The primary reason that concrete bridge decks have cracked more since the mid-1970s is because modern deck concretes have higher early compressive strengths. AASHTO specifications increased the requirement for compressive strength of air-entrained Class A(AE) concrete from 20.7 MPa (3000 psi) to 31 MPa (4500 psi) in the mid-1970s. Recent AASHTO specifications for Class A(AE) concrete require a minimum 28-day compressive strength of 27.6 MPa (4000 psi). These newer concretes have higher cement contents, higher paste volumes, higher moduli of elasticity, higher hydration temperatures, and lower creep. The minimum deck concrete strengths should be evaluated in design, and specifications should not allow high compressive strengths, especially at early ages. Typically, concrete used in bridge decks should have 28-day compressive strengths between 21 and 28 MPa (3000 and 4000 psi). Later age strength levels, such as 45- or 60-day compressive strengths, can also be required. Low-strength concretes can be made durable and resistant to chloride ingress by proper mix proportioning; the use of mineral admixtures, low water-cementitious ratios, air entrainment, and corrosion-inhibiting admixtures; and the application of surface sealers or overlays.

## MIX PROPORTIONS

Mix proportions can significantly affect concrete properties and transverse deck cracking. Transportation agencies should carefully evaluate and select mixes less prone to cracking.

### Cement Content

Transportation agencies should specify concrete with a low cement content for bridge decks. As discussed, reducing the cement content typically increases creep, reducing shrinkage and thermal stresses and the risk of transverse deck cracking. Reducing the cement content also reduces temperatures during early hydration, reducing the thermal stresses that become locked into the concrete during hardening and the risk of early and later transverse cracking. A temperature rise of approximately 7 to 8°C (13 to 15°F) occurs per 45 kg (100 lbs) of portland cement per 0.76 cu m (1 cu yd) of concrete, starting at moderate construction temperatures.

The restrained ring tests indicated that increasing the cement content quickened cracking. Also, many other researchers found that higher cement contents increased deck cracking (*10,14,37*).

ACI Committee 345 (*38*) recommends a minimum cement content of 335 kg/m$^3$ (564 lbs/yd$^3$) and a maximum water-cement ratio of 0.45 for bridge deck concrete. Current AASHTO specifications require a minimum cement content of 362 kg/m$^3$ (611 lbs/yd$^3$) and a maximum water-cement ratio of 0.45. These high cement contents often produce much higher concrete compressive strengths than the AASHTO-specified 27.6 MPa (4000 psi). When such a high strength is not required, the cement content should be reduced to reduce the risk or severity of transverse cracking.

For bridge decks, transportation agencies should specify a maximum cement content instead of a minimum cement content. If the maximum aggregate size is increased to 38 mm (1.5 in.), a 306 kg/m$^3$ (517 lbs/yd$^3$) cement content with a similar water-cement ratio should provide the same workability as current AASHTO Class A or A(AE) deck concretes. When necessary, water-reducing admixtures can be used to improve workability.

### Water Content

The restrained ring testing did not find a relationship between total water content and when cracking developed. Another researcher (*24*) also found little correlation between cracking and the water content in concrete decks. The water content is a primary factor establishing shrinkage and creep of concrete. Concretes with high water contents shrink more than concretes with lower water contents, but additional creep often offsets this additional shrinkage.

### Water-Cement Ratio

Reducing the water-cement ratio usually increases concrete strength and decreases free-shrinkage. However, concretes with lower water-cement ratios have higher moduli of elasticity and lower creep, which increase stresses for a given shrinkage strain. Therefore, when used in a bridge deck, these concretes are sometimes more susceptible to transverse cracking. The restrained ring tests found a slight relationship between the water-cement ratio and when cracking occurred, with some concretes with a lower water-cement ratio cracking slightly sooner. For freeze-thaw protection only, water-cement ratios should not exceed 0.45 when bridge decks will be subjected to freezing.

### Aggregate and Cement Paste Content

In general, transportation agencies should specify lean concrete mixes with high aggregate content and low cement paste content for bridge decks. The concrete paste volume is the component of the concrete that shrinks; reducing the paste volume reduces shrinkage. Leaner mixes are thermally less expansive and develop smaller thermal stresses. Leaner mixes are also more economical. However, creep is related to the volumetric content of the cement paste in the mix and leaner concrete mixes typically creep less and will develop larger stresses for a given strain. Reducing the paste volume reduces the shrinkage but also reduces the creep so the influence on deck cracking may be offset.

## AGGREGATE

The ring tests found that aggregate type affected cracking more than any other concrete mix parameter. Disregarding rings containing Type K shrinkage-compensating cement and extended wet curing, all standard concretes cracked within 53 days. Concrete made with one aggregate type cracked at an average age of 20 days, and concrete with the same material proportions but another aggregate type did not develop distinct full-depth cracking even after 280 days.

Other researchers (*24*) agree that aggregate affects bridge deck cracking. Aggregates with high shrinkage characteristics should be avoided. Aggregate type and size influence the strength, modulus of elasticity, shrinkage, and creep of concrete; together these properties can greatly influence cracking. Selection of aggregates for decks should be based on cracking-tendency ring test results.

### Aggregate Size

Transportation agencies should not limit the maximum aggregate size for bridge deck concrete, except when casting around congested reinforcing areas. According to ACI 318-

89 (*39*) guidelines, the maximum aggregate size should be the smaller of one-third the deck thickness, or three-fourths the minimum clear spacing between bars. Using these ACI guidelines, most bridge decks could be constructed with at least 38-mm (1.5-in.) maximum-sized aggregates.

Larger aggregates permit a leaner mix while maintaining workability and reducing thermal stresses during early hydration. Well-graded larger aggregates also reduce shrinkage and bleeding (*20*). The laboratory ring tests involved very limited testing of the effect of aggregate size. For the limited testing, concrete with 19-mm (¾-in.) maximum aggregate behaved similarly to the concrete with 25-mm (1-in.) maximum aggregate.

### Aggregate Shape

The limited laboratory tests indicated that concrete made with crushed aggregate cracked later than concrete made with rounded aggregate. Each of the aggregates had different chemistry and the influence of aggregate shape was not evaluated independently. Concrete with crushed aggregate may perform better in bridge decks than concrete with rounded river gravel.

### Mineralogy

Aggregate mineralogy affects many of the important concrete properties that influence deck cracking. Shrinkage and thermal stresses can be reduced by selecting aggregate with a low modulus of elasticity, low coefficient of thermal expansion, and high thermal conductivity.

Perhaps the most important mineralogy characteristic is the aggregate's modulus of elasticity. Because concrete is mostly aggregate, the concrete modulus of elasticity is largely affected by the aggregate's modulus of elasticity. The concrete modulus of elasticity affects shrinkage stresses and cracking. Generally, deck concrete made with aggregate having a low modulus of elasticity will be less likely to develop transverse deck cracking.

Also because concrete is mostly aggregate, concrete's coefficient of thermal expansion is determined primarily by its aggregate's coefficient of thermal expansion. The concrete coefficient of thermal expansion affects thermal stresses and transverse cracking. Reducing the coefficient of thermal expansion reduces diurnal and early hydration thermal stresses, while matching the thermal expansion rate of the reinforcement and girder minimizes seasonal thermal stresses. Because early hydration and diurnal thermal stresses are typically much larger than seasonal thermal stresses, reducing the concrete coefficient of thermal expansion will reduce thermal stresses in the deck. Typical values of the coefficient of thermal expansion range from as low as $6 \times 10^{-6}$/°C ($3.3 \times 10^{-6}$/°F) to $13 \times 10^{-6}$/°C ($7.2 \times 10^{-6}$/°F) depending on aggregate type (*41*).

Concrete diffusivity is a measure of how readily heat flows through concrete; a larger value indicates quicker heat conduction. The thermal analyses of steel-girder and concrete-girder bridges reveal that nonuniform temperature changes produce larger thermal deck stresses than uniform temperature changes. Concrete decks with higher diffusivity will have smaller temperature gradients than decks with lower diffusivity, and hence lower thermal stresses. According to ACI 207.2R (*40*), diffusivity values for concrete range from 0.072m$^2$/day (0.77 ft$^2$/day) to 0.129 m$^2$/day (1.39 ft$^2$/day) depending on aggregate type.

## CEMENT TYPE

Researchers (*26*) found that decks constructed with Type II cement cracked less than those constructed with Type I cement. Reducing temperatures during hydration reduces tensile stresses and the risk of transverse deck cracking. Concretes with lower heats of hydration reach lower peak temperatures during hydration and develop lower corresponding thermal stresses. For this reason, Type II or Type IV (low-heat-of-hydration) cement should be chosen instead of Type I cement. Type III (high early strength) cement may increase deck cracking because it increases early hydration temperatures and thermal stresses. If Type III cement is required, use of the minimum cement content to meet strength requirements and early fog-mist curing to reduce hydration temperatures is required. It should be recognized that the same types of cement from different manufacturers may react differently.

The general chemistry and fineness of cements has changed during the past 20 years. Some of these changes correspond to changes in the AASHTO Specifications in the mid-1970s that increased the minimum concrete compressive strengths for decks from 20 to 31 MPa (3000 to 4500 psi). Cement producers changed the cement fineness (Blaine) and composition. The most expensive process and the highest cost associated with cement manufacturing is grinding. Beginning in 1970, producers had to grind cement finer to remain competitive even though it was more expensive. The resulting cements had higher sulfate contents to control the faster-reacting aluminates and higher alkali contents because of pollution control requirements and the uses of fuel types, such as trash and old tires, to heat the kiln.

The finer cements and higher sulfate contents increased early strengths, heats of hydration, and the early modulus of elasticity. Lower-strength concretes produced before 1970 typically had 2-day moduli of elasticity of 40 percent of their 28-day value. The 1-day modulus of elasticity of concretes with modern cements is often 65 percent or greater of the 28-day modulus. For example, the deck concrete on the Portland-Columbia Bridge had a 1-day modulus of elasticity of 19,200 MPa ($2.8 \times 10^6$ psi), nearly 80 percent of its ultimate modulus. Modern concretes with such high early modulus values dramatically increase the risk of cracking because of the high stresses that develop as a result of early shrinkage and thermal strains.

Modern cements also appear to have higher heats of hydration. This will aggravate cracking because the concrete will reach higher temperatures and result in more locked-in deck stress as the concrete cools to ambient. The stresses are further aggravated since the concrete has such a high early modulus. The use of coarser cements will reduce the cost of the cement, the energy used to manufacture the cement, the pollution associated with cement manufacturing, and the risk of early deck cracking.

## SHRINKAGE-COMPENSATING CEMENT

Shrinkage-compensating cement holds promise as a means to reduce transverse cracking although field performance has been mixed. Restrained rings with Type K shrinkage-compensating cement developed light surface cracking and did not develop distinct cracks even after 250 days. Concrete rings containing an ettringite-forming expansive additive cracked at an average age of 36.5 days, 16 days later than the control concrete without the additive.

## SILICA FUME

Researchers claimed that silica fume contributes to increased deck cracking (*43,44*). On one steel-girder bridge investigated by the authors, the deck spans having silica fume had significantly more cracking than one control span not having silica fume. Adding silica fume increases temperatures during early hydration, which increases thermal stresses. It also increases the modulus of elasticity, which increases shrinkage and thermal stresses.

One group of researchers (*45*) noted that conventional concrete shrinks very slowly after its initial hydration swelling. The concrete with silica fume did not swell, and shrinkage was immediate. The researchers attributed this high autogenous shrinkage of silica fume concrete to self-desiccation.

Concretes with silica fume bleed less and are more prone to plastic shrinkage cracking. Restrained laboratory rings with silica fume cracked 5 to 6 days sooner than the control concrete without silica fume, despite similar free-shrinkage. Adding silica fume increases the risk or severity of deck cracking.

## WATER REDUCERS

Water reducers are recommended to reduce water and paste volume, and the risk or severity of early cracking. Adding a water-reducing admixture can maintain workability and consolidation without increasing the water content. Restrained ring tests showed that high-range water reducers delayed cracking slightly.

## SET RETARDERS

Retarders are often used to place a deck continuously. They slow the strength gain and reduce temperature gain and the related thermal stresses, thereby reducing the risk of thermal cracking. They will, however, increase the susceptibility of the concrete to plastic shrinkage cracking. To prevent plastic cracking of continuously placed concrete, designers must account for concrete stresses as a result of dead load.

With retarders, good curing is essential to prevent plastic cracking. Some researchers blamed set retarders for deck cracking, but others found no relationship between the use of retarders and cracking (*21*). The Minnesota DOT does not recommend retarders. Other transportation agencies have recommended retarders to reduce early hydration temperatures (*37*). The ring tests indicated that retarded concrete cracked on average about 2 days sooner than the control mix, but the results were too scattered to permit definite conclusions. Retarders are not recommended in winter or cool conditions because they prolong setting time and increase the risk of plastic shrinkage cracking problems.

## SET ACCELERATORS

Accelerators can worsen bridge deck cracking by increasing early shrinkage, temperatures during early hydration, and the early modulus of elasticity. As such, accelerators generally should be avoided in order to reduce the risk of transverse deck cracking. Although concrete rings made with an ASTM C494 Type C/E accelerating admixture cracked slightly (4 days) sooner than the control mix, accelerators can help reduce plastic cracking.

## FLY ASH

Fly ash is being used in increasing quantities. Fly ash reduces the rate of strength gain and early hydration temperatures, and its use has been recommended to reduce the incidence of deck cracking (*37*). However, replacement of 28 percent of cement with a Type F fly ash did not significantly affect the time-to-cracking of the laboratory rings. Replacing cement with mineral admixtures holds promise to reduce early deck stresses and cracking.

## SLUMP

Fresh concrete must be readily compacted and finished. Without proper compaction, reinforcement has less protection against corrosion. Slump should be least 75 mm (3 in.) for adequate compaction and finishing reasons. Water-reducing admixtures can increase the slump to reasonable levels without detrimentally increasing the water content. However, water-reducing admixtures significantly reduce bleeding, making the concrete susceptible to plastic cracking and thereby requiring early water fogging or evaporation retarders.

The restrained ring tests generally did not indicate a relationship between slump and cracking tendency; the no-slump mixes cracked latest, but these mixes required impractical compaction. Several researchers (*20,28,42*) suggested that higher-slump concretes are more susceptible to cracking because they are more prone to settlement cracking that results from inadequate vibration during placement. Other researchers (*26*) found that slump did not affect deck cracking. Excessive high-slump concretes should be avoided, but within typical ranges of 50- to 200-mm (2- to 8-in.) slump does not appear to affect deck cracking.

TABLE 5 Factors affecting cracking

| Factors | Effect: Major | Moderate | Minor | None |
|---|---|---|---|---|
| **Design** | | | | |
| Restraint | ✓ | | | |
| Continuous/simple span | | ✓ | | |
| Deck thickness | | ✓ | | |
| Girder type | | ✓ | | |
| Girder size | | ✓ | | |
| Alignment of top and bottom reinforcement bars | | ✓ | | |
| Form type | | | ✓ | |
| Concrete cover | | | ✓ | |
| Girder spacing | | | ✓ | |
| Quantity of reinforcement | | | ✓ | |
| Reinforcement bar sizes | | | ✓ | |
| Dead-load deflections during casting | | | ✓ | |
| Stud spacing | | | ✓ | |
| Span length | | | ✓ | |
| Bar type—epoxy coated | | | ✓ | |
| Skew | | | ✓ | |
| Traffic volume | | | | ✓ |
| Frequency of traffic-induced vibrations | | | | ✓ |
| **Materials** | | | | |
| Modulus of elasticity | ✓ | | | |
| Creep | ✓ | | | |
| Heat of hydration | ✓ | | | |
| Aggregate type | ✓ | | | |
| Cement content and type | ✓ | | | |
| Coefficient of thermal expansion | | ✓ | | |
| Paste volume—free shrinkage | | ✓ | | |
| Water-cement ratio | | ✓ | | |
| Shrinkage-compensating cement | | ✓ | | |
| Silica fume admixture | | ✓ | | |
| Early compressive strength | | | ✓ | |
| HRWRAs | | | ✓ | |
| Accelerating admixtures | | | ✓ | |
| Retarding admixtures | | | ✓ | |
| Aggregate size | | | ✓ | |
| Diffusivity | | | ✓ | |
| Poisson's ratio | | | ✓ | |
| Fly ash | | | | ✓ |
| Air content | | | | ✓ |
| Slump[†] | | | | ✓ |
| Water content | | | | ✓ |
| **Construction** | | | | |
| Weather | ✓ | | | |
| Time of casting | ✓ | | | |
| Curing period and method | | ✓ | | |
| Finishing procedures | | ✓ | | |
| Vibration of fresh concrete | | | ✓ | |
| Pour length and sequence | | | ✓ | |
| Reinforcement ties | | | | ✓ |
| Construction loads | | | | ✓ |
| Traffic-induced vibrations | | | | ✓ |
| Revolutions in concrete truck | | | | ✓ |

[†] within typical ranges

## AIR CONTENT

The laboratory rings cast without air entrainment did not show a significantly different cracking tendency than the control mixes. However, past research suggests that concretes with more air are slightly less susceptible to cracking than those with less air (*10,32*). Air content is usually specified for freeze-thaw durability requirements; however, to reduce cracking, it may be slightly advantageous to use air-entrained concrete in environments that are not subject to freezing and thawing cycles.

## FIBER REINFORCEMENT

One DOT research project found that fiber reinforcement reduced early cracking (*23*). In Japan, steel fibers are used to reduce deck cracking. Some researchers found that fibers reduced plastic and settlement cracking (*11*). After cracking, fibers can reduce crack widths (*11*). Further investigation of the use of fiber reinforcement is needed, and fibers may prove useful in reducing early plastic cracking and the width of later cracks.

## SUMMARY OF MATERIAL INFLUENCE AND RECOMMENDATIONS

Many material properties will affect the susceptibility of a concrete deck to cracking. Proposed concretes should be tested for cracking tendency and the mix with the lowest tendency should be used. Table 5 includes various material factors in order of their effect.

CHAPTER 9

# CONCRETE PLACEMENT

## INTRODUCTION

Poor construction practice can increase the effects of early transverse cracking. Some researchers (*21*) found that bridges in Pennsylvania built by two contractors cracked much more than bridges built by nine other contractors. Because many construction factors affect transverse deck cracking, transportation agencies must be careful to specify proper construction techniques, and contractors must follow these techniques closely. A list of the various construction-related factors are ranked by group in Table 5.

## WEATHER AND TIME OF PLACEMENT

Weather during placement can affect the early deck cracks (*26*). To reduce cracking, decks generally should be cast in cool weather. Doing so reduces the concrete rate of hydration, which reduces temperatures during early hydration and associated thermal stresses. Placing concrete during mild weather when diurnal temperature cycles are small reduces early thermal cycles and further reduces the risk of early cracking.

The first large stresses develop in a bridge deck during its early hydration. Within the first 24 hours, temperatures in the deck can easily increase 28°C (50°F) or more, which is then followed by similar cooling. During this time, substantial temperature variations can exist along the length and across the width of the bridge, as heat is nonuniformly transferred out of the deck by its supporting girders or other abutting structure. During the first several hours while the concrete is still plastic, it can adjust to these changing temperatures without developing stresses. Afterwards, temperature changes cause stresses in the deck. Curing, shading, concrete components, size, and initial concrete temperature affect these temperatures and corresponding stresses. Good construction practices can reduce these early temperatures and reduce the risk or severity of transverse deck cracking.

For most bridges, placing cooler concrete during cooler weather can reduce the risk or severity of transverse deck cracking. The Florida DOT found the most cracks in decks cast between May and August, and the fewest cracks in decks cast in October and November. Cooler temperatures reduce the concrete rate of hydration and the thermal cycle during early hydration. The concrete temperature during placement should not be much warmer than the ambient air temperature, so that evaporation rate is not increased.

Cracking can also be a problem when concrete is cast during cold (*19,32*) temperatures. This may be related to the slower setting time, which allows greater evaporation while the concrete is plastic. The maximum air temperature at time of casting specified by the transportation agencies ranged from 27 to 35°C (80 to 90°F), and the minimum air temperature ranged from 2 to 10°C (35 to 50°F). Researchers (*32*) have recommended that decks not be cast when air temperatures are colder than 7°C (45°F). Generally, concrete decks should not be cast when air temperatures are cooler than 7°C (45°F) and warmer than 27°C (80°F); in warmer climates, this may require nighttime casting during the summer months.

During hot weather, concrete decks should be placed during the evening or at night. Air temperatures and solar radiation immediately after placement are lower, reducing the concrete rate of hydration and thermal stresses. Also, relative humidities are typically higher and wind speeds are lower in the evening, reducing the risk of plastic shrinkage. Usually, placing the concrete around noon on a sunny day maximizes early temperatures and stresses, and the risk of early cracking. Researchers found more cracking in pours cast during low humidities and high evaporation rates (*28*). Casting at night significantly reduced deck cracking, and late-morning and afternoon pours were most likely to crack (*20,23*). A survey of pavements cured with clear membranes found that cracking occurred predominantly in pavements placed in the morning hours (*46*).

## CONCRETE PLACEMENT TEMPERATURE

To prevent large thermal stresses during early hydration, concrete placement temperatures should be considered. Concrete decks should be cast 5 to 10°C (10 to 20°F) cooler than ambient air temperature, unless air temperatures are below 16°C (60°F), when the concrete temperature should closely match the air temperature. When warm concrete is cast in cool weather, the concrete heats the air immediately above the surface, reducing the humidity of this air layer, which increases concrete evaporation and the risk of plastic shrinkage. The temperature of the concrete when it is delivered to the site should not be more than 5°C (10°F) warmer than the ambient air temperature.

Transportation agencies do not agree on appropriate concrete placement temperatures. PCA (*20*) recommended a maximum placement temperature of 28°C (80°F). Most transportation agencies limit the maximum concrete temperature to 32°C (90°F), but some have lower limits. The agencies generally limit the minimum concrete placement temperature to between 7 and 16°C (45 and 60°F).

To reduce concrete temperatures, concrete suppliers should shade aggregates before mixing, and replace part of the mix water with ice (*23,37*). Also, water misting or sheeting during cool weather and hot weather is important to prevent evaporation. Casting cool concrete, reducing the cement content, placing at night, water misting, and applying white-pigmented curing compounds will help reduce the peak concrete temperatures resulting from hydration and also from early thermal stresses.

## WIND SPEED

When the wind speed over the concrete is 8 km/h (5 mi/h) or less, the evaporation rate and the probability of plastic shrinkage cracking is low. The evaporation rate should be measured at the jobsite, and wind breaks and fogging should be used during periods of high evaporation to prevent plastic drying shrinkage. Construction specifications should require wind breaks and immediate water fogging when the evaporation rate exceeds 1 $kg/m^2/hr$ (0.2 $lb/ft^2/hr$) for normal concretes, or 0.5 $kg/m^2/hr$ (0.1 $lb/ft^2/hr$) for concretes with high cement contents, silica fume, HRWRAs, or other constituents that reduce the bleed of the concrete. Bleed should be tested to assess susceptibility to plastic shrinkage cracking.

## PLACEMENT SEQUENCE

Placement sequence can affect deck cracking in continuous-span bridges, and some transportation agencies recommended sequenced pours. The New Mexico DOT believes that continuous placement sequences promote deck cracking. Other research has found that pour length and sequence did not influence transverse cracking (*23,36*). Transverse deck cracking occurs in continuous-span bridges with and without sequenced pours.

By placing the center portions of spans first in a continuous-span bridge, negative bending and tensile stresses over the interior supports are reduced, as is the risk of transverse deck cracking. Proper sequence is important, but sequence is not a primary cause of early cracking and sequencing pours can extend construction time.

## FINISHING

Proper finishing is essential to reduce the risk or severity of transverse deck cracking. The contractor should thoroughly vibrate and strike the concrete with a mechanical screed. The concrete should then be smoothed using a float, if necessary. Final floating should be delayed until after the early bleeding to prevent crusting of the surface that traps bleed water and weakens the surface, making it susceptible to rapid scaling. Caltrans (*26*) found that late finishing and hand finishing increased cracking. New Mexico DOT also reported that early finishing reduced cracking. Early finishing reduced the number and width of cracks, and double-floating decreased cracking even further (*26,48*).

There are many mechanical finishing machines, and their effects on early cracking vary. Finishing machines that can rapidly consolidate and finish the concrete with the minimum amount of manipulation are best suited for decks.

To reduce surface drying before curing, the contractor should mist the deck surface using fog nozzles specifically designed for concrete placement or apply evaporation-reducing films. Fogging should commence immediately after strike off. Water should never be sprinkled directly onto the new concrete surface and worked into the surface.

## TINING

Texturing of most bridge decks is done by dragging a tining broom transversely across the deck. A tined surface drains water better than a smooth surface, but it provides a noisier ride. Because tining can delay curing, many states recommend mechanical grooving instead of tining so that curing can start sooner. Without tining, evaporation retarders and curing compounds can be applied sooner, and wetted fabric coverings can be applied sooner because of damage to the tinings is not a concern. Mechanical grooving also damages the surface of the concrete less than rake tining and provides more uniform and durable grooves. When bridge decks are cast during high evaporation conditions, curing as soon as possible followed by mechanical grooving can reduce deck cracking.

## VIBRATION OF FRESH CONCRETE

Effective consolidation improves all important properties of concrete, and problems of under-vibration are more widespread than those of over-vibration (*32*). Areas of under-vibration are more prone to cracking, (*24*) and some surveyed agencies believe inadequate vibration is a major cause of cracking (*23*).

Concrete settles during its placement, vibration, and finishing. If reinforcing bars or formwork prevents the concrete from settling in localized areas, voids and cracking occur in adjacent areas. Figure 6 shows the relationship of concrete slump, cover, and bar size to settlement cracking.

Especially in hot windy weather, surface crusting of the concrete can occur. This may promote early finishing that may trap bleed water and result in surface scaling and poor surface durability. Vibration must occur late enough to ensure close contact with the reinforcing steel and re-vibration may be necessary if bleeding is prolonged. Very early plastic shrinkage cracks can be closed when the concrete is re-vibrated while it is still plastic.

At least three vibrators are recommended for placement rates of 22 $m^3$/hr (30 $yd^3$/hr) or higher. Vibrator frequency, size, and time of insertion can generally vary without affecting consolidation (*32*).

## CONSTRUCTION LOADS

Early construction loads can cause cracking (*26*). Heavy construction machinery or stacked supplies can overload the deck and cause cracking, especially over supports of continuous-span structures when the concrete is young. However, construction loads are not a large cause of transverse deck cracking.

## TRAFFIC-INDUCED VIBRATIONS

Research shows that traffic-induced vibrations before or during concrete hardening do not cause cracking (*21,34,35*). Deflections associated with the vibrations are too small to damage the concrete.

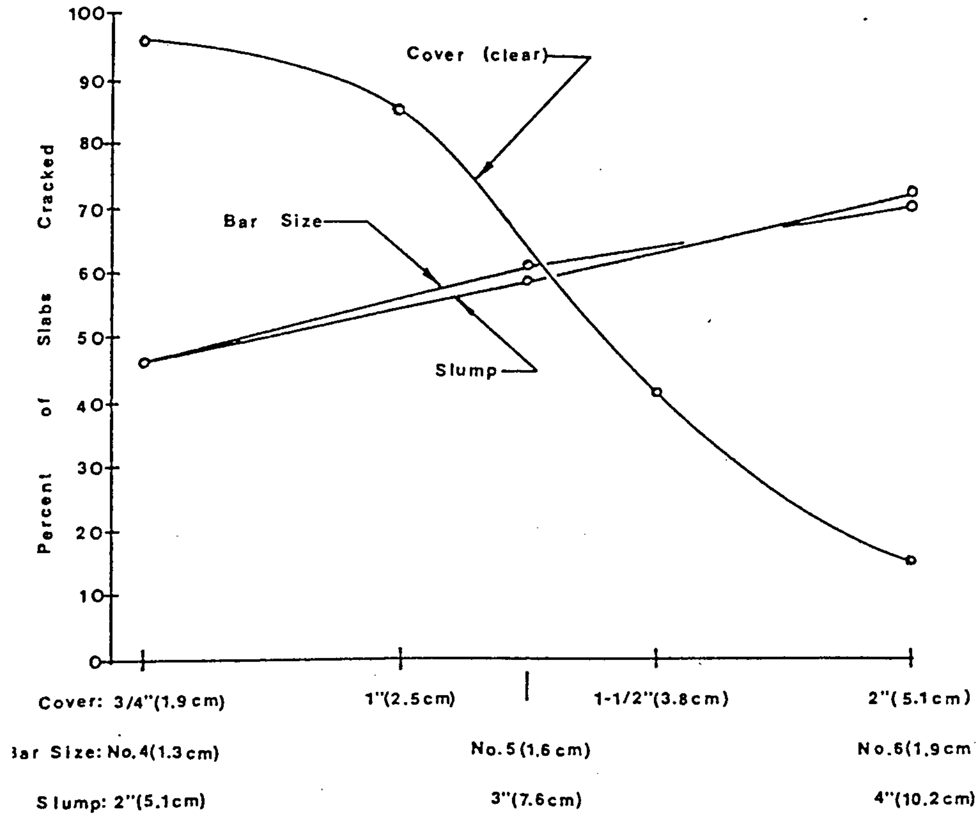


*Figure 6. Cracking as a function of concrete slump, clear cover, and bar size. (Bakr, Cady, and Carrier)*

## REVOLUTIONS IN CONCRETE TRUCK

Research by one transportation department did not indicate a correlation between the number of revolutions of transit mix trucks and deck cracking (*26*). Mix drums should be turned at least 70 to 100 revolutions after all ingredients are added to ensure thorough mixing; however, mixing is affected by the concrete mix itself and the truck. Excessive mixing should be avoided because of the grinding of soft aggregates and the increase in temperature of the concrete.

# CHAPTER 10
# CURING

## INTRODUCTION

Curing can affect transverse deck cracking. PCA (*47*) defines curing as the maintenance of a satisfactory moisture content and temperature in concrete during some definite period immediately following placing and finishing so that the desired properties may develop. Curing significantly affects durability, strength, watertightness, abrasion resistance, volume stability, and resistance to freezing and thawing and deicer salts. Improper curing can reduce compressive strengths as much as 40 percent.

The first several days are critical to the strength and durability of the deck. Fogging immediately after strike-off and evaporation retarders can significantly reduce very early plastic cracking (*26*). Early moist curing, using wet burlap instead of a curing compound, can reduce cracking. Cracking is worse when water curing is delayed.

Most transportation agencies cited ineffective curing as the most common cause of excessive transverse deck cracking (*20,48*). However, the survey responses revealed that there is no standard method used to cure bridge decks and curing practices vary widely. Many agencies allow only membrane or curing compounds, while others require curing compounds combined with long-term wet curing. Some specifications allow the contractor to select curing methods; however, this option may result in the selection of membrane or curing compounds that are less expensive and easier than long-term wet curing but that are less effective.

## PLASTIC SHRINKAGE CRACKING

Plastic shrinkage cracks develop while the concrete has not yet hardened, when weather conditions (air temperature, humidity, wind velocity) cause rapid evaporation that exceeds the bleed of the concrete. Plastic cracking occurs in all directions, and its causes and preventions are well known. The plastic cracks may be as wide as 6.3 mm (¼ in.) where they initiate, but they typically are shorter than 0.6 or 0.9 m (2 or 3 ft) and shallower than 50 to 75 mm (2 to 3 in.). Plastic shrinkage cracking can be severe, and plastic cracks can allow liquid and gas penetration into the concrete. They are seldom structurally significant, and stresses rapidly dissipate.

Certain concretes are more susceptible to plastic shrinkage cracking than others. This cracking is often worsened by low water-cement ratios and admixtures such as latex modifiers, superplasticizers (HRWRAs), air entrainment, and silica fume. Latex modifiers and HRWRAs reduce the water content and bleeding. Similarly, concretes with silica fume require HRWRAs to offset the reduced workability caused by the addition of silica fume. These newer types of concrete typically do not bleed or have minimal bleed.

The obvious solutions to reduce plastic shrinkage cracking are to reduce the evaporation rate or increase concrete bleed. It is not possible to significantly increase bleeding with modern deck concretes. The evaporation rate can be reduced by sunscreens, windbreaks, fog mist, and monomolecular curing films. Polyethylene sheeting or other impermeable covers are the most effective means to avoid losing bleed water, but such covers can be cumbersome on a bridge deck. The most cost-effective method to prevent plastic cracking usually involves applying a mist to the concrete surface from the upwind side of the work. This must be done with a commercial grade fog nozzle that produces a mist too fine to damage the concrete surface and has broad coverage immediately after screeding. Care must be exercised so that the fogged water does not become part of the concrete during finishing. Wetting forms before the concrete is placed is also helpful.

It is well-known that high evaporation occurs on hot days, but it is not generally understood that higher evaporation can occur during cooler weather. When warm concrete is cast during winter conditions, the warm concrete heats the air immediately above the surface, reducing its relative humidity. This warm moist air is replaced by cold dry air that quickly warms up and absorbs more moisture, aggravating drying and increasing the risk or severity of plastic shrinkage cracking. Also, concrete placed during cooler weather remains plastic longer and is affected for a longer time by evaporation.

A nomograph relating air temperature, relative humidity, concrete temperature, and wind speed to evaporation was developed (*9*). Reportedly, plastic shrinkage cracking of ordinary concretes is less likely when the evaporation rate does not exceed 1 kg/m$^2$/hr (0.2 lb/ft$^2$/hr). However, lower values may be appropriate for modern concretes, especially those with high cement contents or additives such as superplasticizers or silica fume.

The nomograph (*9*) can be used to estimate the safe relative humidities at which the evaporation rate does not exceed 0.1 kg/m$^2$/hr (0.2 lb/ft$^2$/hr) for various air temperatures, concrete temperatures, and wind speeds. To examine the dangers of casting warm concrete in cool weather, consider concrete temperatures of 27°C (80°F), and moderate wind speeds of 16 to 24 km/h (10 and 15 mi/h). Under these conditions, concrete should not be placed at air temperatures cooler than 4°C (40°F). Even worse, 32°C (90°F) concrete should not be placed in air temperatures of 16°C (60°F) or cooler unless provisions are made to reduce the evaporation rate.

A "gentle breeze" is typically 16 km/h (10 mi/h) and a "moderate breeze" is 24 km/h (15 mi/h). With wind speeds above a gentle breeze, strict control of temperature, humidity and placement temperatures are required to reduce evaporation rates below 1 kg/m$^2$/hr (0.2 lb/ft$^2$/hr).

Conversely, when wind breaks can reduce the wind speed over the concrete to less than 8 km/h (5 mi/h), there is a low probability of plastic shrinkage cracking. Construction specifications should require windbreaks and immediate water fogging when the evaporation rate exceeds 1 kg/m$^2$/hr (0.2 lb/ft$^2$/hr) for normal concretes and 0.5 kg/m$^2$/hr (0.1 lb/ft$^2$/hr) for concretes susceptible to plastic cracking such as those containing latex emulsions, silica fume, or HRWRAs.

## CRACKING-TENDENCY TESTS

The restrained concrete ring tests suggest that curing affects high-cement-content, low water-cement ratio concretes more than other concretes. Extended wet curing of 60 days did not significantly change the time-to-cracking of ring specimens with 278 kg/m$^3$ (470 lb/yd$^3$) cement and a water-cement ratio of 0.5. However, extended wet curing did delay cracking 9 days (to an age of 21 days) when the mix contained 501 kg/m$^3$ (846 lb/yd$^3$) cement and a water-cement ratio of 0.35. The time-to-cracking was calculated from when the wet curing was stopped. Long-term wet curing of low water-cement ratio concrete delayed cracking but did not prevent it.

## CONTINUOUS MOIST CURING

A continuous water mist, water ponding, or saturated coverings such as wet burlap provide continuous moist curing. The contractor should pre-wet an absorbent covering prior to placing it on the concrete, so that it does not wick moisture from the concrete; however, a saturated covering can be heavy and difficult to handle. Contractors sometimes pre-wet the deck and then immediately wet the fabric after it is placed. With absorbent coverings, the contractor must take special care to keep the coverings wet and to prevent wind from uncovering the edges. Wet coverings can also reduce concrete temperature and early thermal stresses.

## MEMBRANE CURING

Membrane curing consists of spraying a compound onto the surface of the concrete to reduce drying. It can be applied sooner than moist curing blankets, and its effectiveness does not end abruptly. Curing compounds will not cure the deck as well as moist curing, because they allow some moisture loss, and curing compounds should not be used without moist curing.

Modern curing compounds usually meet or exceed the requirements of ASTM C309 (*38*), which allows up to 0.55 kg/m$^2$ (0.11 lbs/ft$^2$) moisture loss during the 72-hr test. This value is low, but membrane curing does permit some drying. Also, contractors often do not apply the membrane curing uniformly, and many areas cure poorly.

For the best deck curing, the contractor should apply a curing compound as soon as possible after finishing, followed by continuous wet curing. The concrete surface should be damp when the curing compound is applied. The curing compound will reduce the initial concrete drying and slow the drying after the wet cure is stopped. White-pigmented curing compounds should be used in hot sunny weather to reduce absorbed radiation and reduce concrete temperature.

Sheet curing by placing plastic sheeting over the concrete is effective only if the concrete is kept continuously wet. Plastic sheeting can help keep wetted burlap or fabric moist, but should not be relied upon as the only method of curing since it can increase concrete temperatures due to trapped heat and solar radiation.

## OPTIMUM CURING

Curing practices vary widely among states. Standardized curing should produce better bridge decks. Better curing and casting procedures will reduce transverse deck cracking but will not prevent it in all bridges. Currently AASHTO prescribes curing procedures for precast concrete, but not for cast-in-place bridge decks. Optimum curing for bridge decks includes the following:

- Use of windbreaks when the evaporation rate exceeds 1 kg/m$^2$/hr (0.2 lb/ft$^2$/hr) for normal concretes, and 0.5 kg/m$^2$/hr (0.1 lb/ft$^2$/hr) for concretes with high cement contents, low water-cement ratios, silica fume, or HRWRAs;
- Use of a fog nozzle water spray to cool the concrete during placement and finishing;
- Application of mist water or a monomolecular film immediately after screeding, and as necessary thereafter;
- Application of a white-pigmented curing compound uniformly in two directions when the bleed water diminishes but before the surface dries;
- Moist curing with either fog or wet curing, especially when concretes with high cement contents and low water contents are used; (When the concrete can resist indentation, the deck should be covered with pre-wetted burlap, and kept wet by continuous sprinkling or by covering the burlap with plastic sheeting and periodic sprinkling. This curing should last at least 7 days [preferably 14 days] and longer during cool weather.)
- Application of a membrane curing compound when wet curing stops to slow the rate of drying if not applied previously; and
- Grooving by diamond saw cutting instead of rake tining, so that the wet burlap can be quickly applied.

CHAPTER **11**

# CRACK REPAIR

## CRACK CHARACTERISTICS

Transverse deck cracks may reduce serviceability or affect structural strength, or both. A qualified engineer must analyze cracks that may affect the strength of the bridge, and structural repairs must be made to those that do. Cracks in areas subjected to deicing or aggressive solutions can reduce serviceability, and these cracks should be filled to prevent infiltration into the cracks.

When deck cracking is caused by internal expansive chemical reaction, such as alkali-silica reaction, repairs may not be effective and complete deck replacement may be warranted. When cracking is from corrosion of embedded reinforcing or cyclic freezing, usually it is best to remove and replace the damaged concrete instead of repairing the cracks.

Most early transverse deck cracks are uniformly spaced and have small to moderate widths, 0.03 to 0.25 mm (0.001 to 0.010 in.). Bonding or sealing these cracks will improve the serviceability of the deck, especially in deicing areas.

## SELECTION OF CRACK REPAIR METHODS

After the cause of deck cracking is determined, a repair method should be chosen with consideration to application ease, durability, life-cycle cost, available labor skills and equipment, local experience, and appearance of the final product. Usual methods for repair include epoxy injection, high molecular weight methacrylate (HMWM) topical treatment, silane and siloxane sealers, routing and sealing, and stitching.

Epoxy injection is commonly used to repair large distinct cracks, and HMWM is commonly used to repair decks with many fine cracks. Trial sections are recommended and coring should verify penetration. Grinding or abrasive blasting readily removes excess material hardened on the surface.

## EPOXY RESINS

Epoxy injection consists of drilling holes along the cracks, installing entry ports over these holes, and injecting epoxy under pressure. This work is labor intensive and time consuming, and generally limited to decks with few large cracks. Epoxy is the most common resin for pressure injection, although other resins can be used. Epoxy resins can be injected into racks wider than 0.05 mm (0.002 in.). Certain low-viscosity epoxy resins are available for gravity application to cracks. The authors' experience has shown that these resins can penetrate into fairly wide cracks, typically wider than 0.50 mm (0.020 in.), but do not penetrate into finer cracks as well as HMWM resins.

ACI Committee 503R (*49*) and 224 (*4*) reports contain detailed information on epoxy injection. ASTM Standard C881, (*15*) Type I, low-viscosity grade epoxy is suitable for most crack injection. Mixing can be done by batch or by continuous machine mixing. The maximum pressure of the injection must be chosen carefully by an experienced contractor.

## HIGH MOLECULAR WEIGHT METHACRYLATE RESINS

The HMWM resins were developed for topical treatment of bridge decks with many narrow cracks (*50*). These low-viscosity resins (8 to 20 cps, similar to diesel fuel) readily flow by gravity into deck cracks narrower than 0.02 mm (0.001 in.). HMWM resins are less effective at filling cracks wider than 0.25 mm (0.010 in.) because these wider cracks have lower capillary forces.

The high solvent capacity of HMWM enables it to bond through lightly contaminated surfaces. Curing compounds and asphaltic materials should be removed before treatment because the resin will solvate them and thicken, causing poor final properties. The cracks must be dry when the HMWM is applied, because water will prevent the concrete from penetrating into the concrete and will dilute the resin. HMWM has low volatility and does not readily evaporate. Standard methyl methacrylate (MMA) resins are very volatile and are not suitable for filling cracks.

A metallic drier and peroxide catalyze HMWM monomers to initiate polymerization. The resin is then swept, squeegeed, or sprayed on the cracked concrete at a rate of approximately 0.4 L/sq m (1 gal/100 sq ft). The resin flows into the cracks and polymerizes, bonding crack surfaces. Dry sandblasting sand should be broadcast into the resin on the deck surface before the resin hardens, to improve skid resistance.

The performance of different HMWM resins can vary. Product selection should be based on satisfactory use in similar applications, and a trial application is recommended for large jobs. Generally, the resin will perform well if it is applied when the concrete and air temperatures are between 7°C (45°F) and 32°C (90°F). Special formulations of HMWM resins are available for use during cold or hot weather.

HMWM resins are brittle and are abraded by traffic. They also do not repel water as well as penetrating sealers such as silanes. Because HMWM resins are compatible with silane sealers, the HMWM resin can be applied after the deck surface has been sealed with the silane sealer so the deck surface will be sealed and the cracks will be filled and bonded.

## SEALERS

ACI 116R (*51*) defines a sealer as "a liquid that is applied as a coating to a surface of hardened concrete to either prevent or decrease the penetration of liquid or gaseous media, for example, water, aggressive solutions, and carbon dioxide, during service exposure." Different materials have been used to coat or seal bridge decks, with varying effectiveness.

Penetration sealers may be used to coat the surfaces of cracks in decks, making them water repellent. Further research is necessary to determine if these sealers can be effective on deck cracks where the surface of the cracks is subjected to high tire pressures.

*NCHRP Report 244* (*52*) found five categories of effective sealers: polyurethanes, MMA, certain epoxy formulations, relatively low-molecular-weight siloxane oligomers, and silanes. Urethanes and epoxies are the most common film-forming sealers. Silanes, siloxane oligomers, and methyl methacrylate are the most common penetrating sealers. *NCHRP Report 244* also evaluated a silane sealer on deeply cracked, reinforced concrete slabs subjected to a rigorous 48-week cyclic wet/dry corrosion test. These silane-treated cracked specimens with 0.25 mm (0.010 in.) wide cracks showed significant corrosion reduction as indicated by very low half-cell potentials, when compared to the cracked reinforced concrete slabs without the silane treatment. Barrier sealers that rely on maintaining a continuous film are not appropriate for bridge decks because they quickly abrade and may reduce skid resistance.

Silane and siloxane are both derived from the silicone family. When catalyzed by moisture, these silicon materials react with the silica available in concrete to form a hydrophobic siloxane resin film that repels water without loss of vapor transmission properties. Silane and siloxane sealers are effective because their very small molecules infiltrate the micropores and capillary structure of the concrete. Penetration depth is mainly related to the amount of active solids applied. Newer volatile organic content (VOC)-compliant products include 100 percent silanes and water-dispersed silanes, both of which can penetrate as well as their predecessors. Further research is needed to determine the effectiveness of sealers on preventing water ingress into deck cracks subjected to tire pressures.

**MEMBRANES AND OVERLAYS**

Membranes and overlays can be applied in severe cases of deck cracking. Typical overlays include latex-modified portland cement concrete, silica fume-modified portland cement concrete, epoxy resin, and polyester resin concretes. Membranes can be applied but must be protected by covering with a wearing surface, typically an asphalt concrete overlay. Membranes and overlays (1) are much more expensive than repairing cracks with epoxy, HMWM, or penetrating sealers; (2) are more difficult to apply; and (3) have potential durability problems.

CHAPTER 12

# SUMMARY

## STRAINS, RESTRAINT, AND STRESSES

Strains, restraint, and stresses cause transverse bridge deck cracking. Shrinkage and temperature changes in the deck are the primary causes of transverse deck cracking. Bridge girders restrain the deck strains associated with shrinkage and temperature change, causing stresses to develop in the deck. The concrete properties affect these stresses. Careful design, material selection, and construction techniques can reduce the shrinkage and thermal strains and stresses that cause transverse deck cracking.

## DESIGN

Design can reduce the restraint on the deck and the risk of transverse deck cracking by careful selection of girder type and size. Equations have been developed to enable bridge designers to calculate shrinkage and thermal stresses in bridge decks, so that they can evaluate and modify designs to reduce these stresses and the risk of transverse deck cracking. Generally, the risk of transverse deck cracking increases when girder size increases, when thinner decks are used, when spans are continuous instead of simply supported (especially with concrete girders), and when steel girders are used instead of concrete girders if spans are simply supported.

Noncomposite design can reduce restraint of the deck and stresses; however, this may not eliminate transverse deck cracking, because some restraint inevitably will develop from friction at the deck-girder interface. Form oil or other friction reducers at the interface can reduce the restraint in noncomposite bridges. Noncomposite design often requires larger girders, increasing construction cost.

To eliminate the risk of transverse deck cracking, decks can be precast and prestressed, and connected to the girders after the initial shrinkage and thermal movements. However, this construction is usually more expensive than traditional casting, and precast decks may not be practical for many bridges.

Designing the longitudinal post-tensioning to produce compressive stresses in the deck will generally decrease the risk of severity of transverse deck cracking, but decrease the flexural efficiency of the system. When it is not practical to design a post-tensioned deck without flexural tensile stresses, tensile stresses should be minimized and deck reinforcement details in those areas should be selected to control the cracking that may develop.

Concrete with acceptable minimum early strength should be used. The compressive strength of concrete decks should be specified at later ages than the normal 28 days. The actual "in-place" compressive strengths should be kept at levels similar to the "specified" strength. Lower-strength concrete typically creeps more than higher-strength concrete, reducing the stresses that develop from shrinkage and thermal strains, and reducing the risk or severity of transverse deck cracking.

Designers can control transverse deck cracking with additional longitudinal reinforcement in the concrete deck. As a minimum, longitudinal size 10M (slightly smaller than a No. 4 bar) bars should be placed at a maximum spacing of 150 mm (6 in.); deck performance is expected to improve with smaller reinforcement spacing. In marine or deicing areas, decks should be reinforced with epoxy-coated steel or corrosion-resistant reinforcement such as stainless steel. Such reinforcement provides significant protection from corrosion, even with cracks, and will extend the service life of the deck while decreasing maintenance.

## MATERIAL SELECTION

Concrete properties generally are the most important factors affecting transverse deck cracking. Concrete properties control the shrinkage and thermal strains that cause stresses, and the relationship between strain and stress. Concrete properties are easier to change than design parameters. Ideally, concrete used in bridge decks should have the following properties to reduce cracking: low modulus of elasticity, high creep, low coefficient of thermal expansion, low heat of hydration, and high thermal conductivity. Doing so reduces shrinkage and thermal strains, and the stresses developed.

The ring test developed for this project should be used to evaluate and develop concrete mixes that are less likely to produce cracking. Generally, mix designs should have a low cement content and large aggregate content, large well-graded crushed aggregate, aggregate with a low modulus of elasticity, aggregate with a low coefficient of thermal expansion, aggregate with high conductivity, and Type II or IV cement. Also, use coarse cements that result in a low 1-day modulus of elasticity and lower heat of hydration. Shrinkage-compensating cement may reduce transverse cracking although field performance results are mixed.

Deck concrete should contain the largest possible aggregate size. Using ACI 318-89 (*39*) guidelines, the maximum aggregate size is the smaller of one-third the deck thickness, or three-fourths the minimum clear spacing between bars.

Instead of specifying a minimum cement content, a maximum cement or cementitious content should be specified for concrete used in bridge decks. Water-reducing admixtures can improve workability when concrete has a low cement content.

## CONSTRUCTION TECHNIQUES

Construction can affect transverse deck cracking. Careful construction practices should be required to reduce the risk of transverse deck cracking.

The first large stresses in a concrete deck can develop during the first 12 to 24 hrs, when temperatures change rapidly from early hydration. Reducing the concrete temperatures during this cycle will reduce early stresses. This can be done by placing concrete during cooler weather, placing cooler concrete, misting the concrete during placement and wet curing, and shading the deck.

When possible, concrete bridge decks should be placed during early or mid-evening. Doing so reduces the hydration temperatures, the thermal stresses that develop, and the risk or severity of transverse bridge deck cracking. Also, air humidities are usually higher in the evening, and nighttime placements reduce the risks of early shrinkage.

The evaporation rate should be measured during placement. During periods of moderate to high evaporation, the contractor should install windbreaks to reduce wind speed over the concrete, and fog mist should be applied immediately after screeding and thereafter when needed to balance evaporation. Water should never be sprinkled directly onto the concrete surface and worked into the concrete.

Proper finishing is essential to reduce the risk or severity of transverse deck cracking. The contractor should thoroughly vibrate and strike the concrete with a mechanical screed. It should then be smoothed using a float, if necessary. Finishing should be completed as soon as practical without finishing the bleed water into the surface. Wet curing should start as soon as possible, even if a curing compound was applied. The contractor should rework and close plastic shrinkage cracks while the concrete is still plastic. Re-vibration of the concrete may be beneficial.

Curing should start as soon as possible. If the contractor does not mist fog the concrete during placement, he or she should apply an evaporation retarder and curing compound as soon as possible. Extending concrete wet curing increases the strength gain, and decreases the rate of shrinkage and final shrinkage.

Diamond grooving, instead of rake tining, may be beneficial because curing compounds and wetted blankets can be applied sooner. For bridges subjected to deicers, contractors should be required to repair all visible deck cracks 6 months to 1 year after construction.

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# PROPOSED STANDARD METHOD FOR TESTING CRACKING TENDENCY OF CONCRETE

# PROPOSED STANDARD METHOD FOR TESTING CRACKING TENDENCY OF CONCRETE

This proposed testing method is the recommendation of NCHRP Project 12-37 staff at Wiss, Janney, Elstner Associates, Inc. It has not been approved by NCHRP or any AASHTO committee or formally accepted for the AASHTO Specifications. It is submitted for trial use and comment to engineers engaged in design of concrete bridge decks.

The following material contains a proposed method for testing the cracking tendency of concrete. The method is proposed for adoption by AASHTO and inclusion as part of *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing.*

## DRAFT

## PROPOSED STANDARD METHOD FOR TESTING CRACKING TENDENCY OF CONCRETE

### Scope

This method covers the determination of the cracking tendency of restrained concrete specimens. The procedure determines the effects of variations in the properties of concrete on the time-to-cracking of concrete when restrained. The procedure is comparative and not intended to determine the time of initial cracking of concrete cast in a specific type of structure. Actual cracking in service depends on many variables including construction methods, bridge type, curing methods, degree of restraint, hydration effects, and environmental factors. The method is useful for determining the relative likelihood of early concrete cracking and for aiding in the selection of concrete mixtures that are less likely to crack. The test method may also be modified to evaluate other factors that may affect cracking such as curing time, curing method, evaporation rate, or temperature.

This standard may involve hazardous materials, operations, and equipment, and it does not purport to address all of the safety problems associated with its use. It is the responsibility of the user to establish appropriate safety and health practices and determine the applicability of regulatory limitation before use.

### Referenced Documents

| | |
|---|---|
| AASHTO T126: | *Making and Curing Concrete Test Specimens in the Laboratory* |
| AASHTO T160: | *Length Change of Hardened Hydraulic Cement Mortar and Concrete* |
| AASHTO M210: | *Apparatus for Use in Measurement of Length Change of Hardened Cement Paste, Mortar, and Concrete* |

### Significance and Use

This test can determine the effects of concrete variations on cracking tendency. These variations might include aggregate source, aggregate gradation, aggregate bond, cement type, cement content, water content, mineral admixtures, silica fume admixtures, fiber reinforcement, or chemical admixtures. The test method measures the strain in a steel ring as a surrounding concrete ring shrinks. The time-to-cracking of the concrete ring is measured as the time when an abrupt drop is seen in the steel-ring strain. Simple visual monitoring of the time-to-first-cracking may also be useful, but it is less accurate than monitoring strains and large errors may result with visual monitoring. The test can also evaluate environmental and construction factors by modifying the test environment or curing procedures.

Other concrete tests that may be related to cracking tendency are unrestrained free-shrinkage, compressive strength, tensile strength, elastic modulus, Poisson's ratio, and creep.

## Apparatus

*Steel Ring*. The standard steel ring shall have a wall thickness of 9.5-mm ± 0.4-mm (½-in. ± 1/64-in.) thick, an outside diameter of 305 mm (12 in.), and a height of 152 mm (6 in.). (See Figure 1). Theoretical elastic analysis indicates that decreasing the steel thickness increases stresses in the steel significantly but only slightly affects concrete stresses. Bond strain gages at four equidistant mid-height locations on the interior of the steel ring. Structural steel pipe conforming to ASTM A501 or A53 12-in. extra strong pipe with an outside diameter of 324 mm (12¾ in.) and wall thickness of 13 mm (½ in.) may be substituted.

*Data Acquisition*. The data acquisition unit should be compatible with the strain instrumentation and automatically record each strain gage independently. Often when cracking occurs only one or two gages indicate significant strain relief. Monitor a strain gage mounted on an unstressed piece of steel to allow for temperature compensation of the steel ring-strain gage readings. Record ambient temperatures.

*Forms*. The forms should be nonabsorbent. Fabricate the base forms of resin-coated or polyethylene-coated plywood to minimize friction restraint of the concrete. Thin 3-mm (⅛-in.) polyethylene sheeting works well as the outside radius form. Secure the steel ring to the base with a central hold-down device during casting. Coat the steel ring surface in contact with the concrete with a release agent such as paraffin wax dissolved in solvent or other suitable form release agent. Do not use form release agents on the exterior form.

*Curing*. Wet cure the top surface, using prewetted burlap covered with plastic. After stripping the form, seal the top of the concrete ring with rubber matting or plastic sheeting designed to eliminate drying from the top, and break the bottom of the ring specimen loose from the base form, but keep the base form in place.

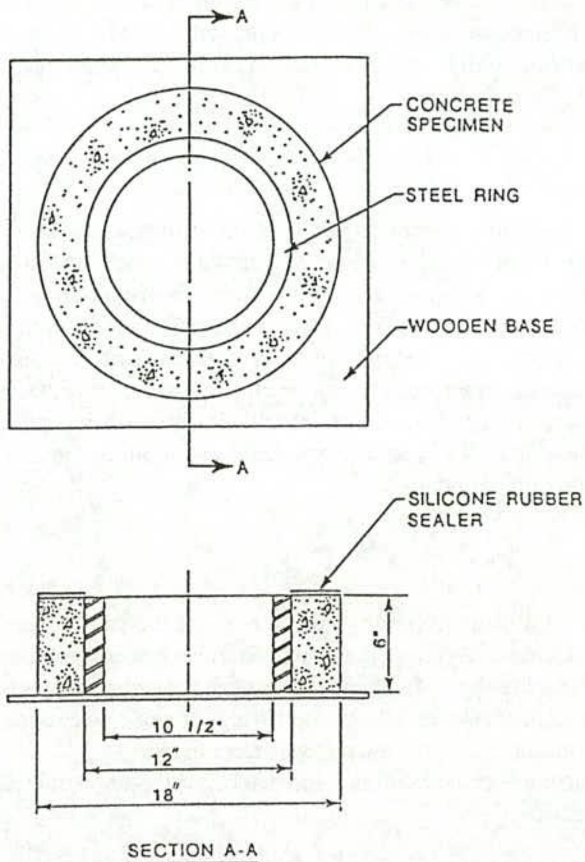
*Curing and Test Room*. After wet curing, store the samples in a controlled-environment room with a constant air temperature of 21°C ± 1.7°C (70° ± 3°F) and a relative humidity of 50 ± 4 percent. Note and record the evaporation rate near the ring surfaces as described in AASHTO T160.

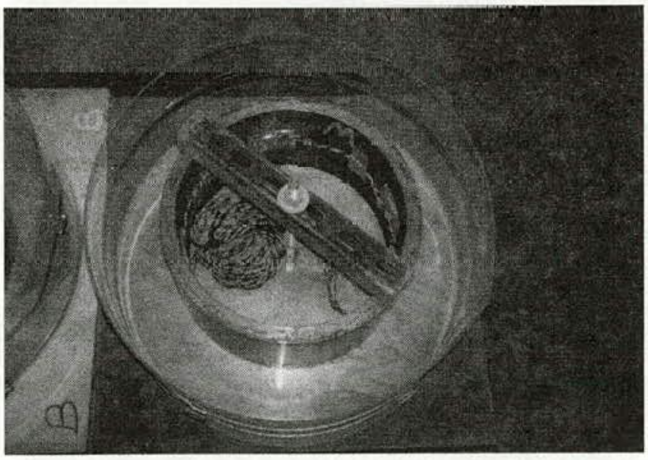
*Miscellaneous*. Test the unrestrained free-shrinkage of companion concrete prisms in accordance with AASHTO T160.

## Test Ring Specimens

Make and cure the test ring specimens following the applicable requirements of AASHTO T126 and M210. Cast at least two concrete rings for each batch. Rod the concrete into the molds in three equal lifts, using a round-nosed rod with a diameter of 16-mm (⅝-in.). Rod the concrete equidistantly 75 times per layer, ensuring that the rod slightly penetrates into the previous layer. Spade the inside and outside surfaces of the mold after each lift to eliminate large voids along the form faces. Lightly tap the base of the mold with a rubber mallet to close any holes left by rodding and to release any large air bubbles; do not tap on the exterior radius of the molds or on the steel ring. While not recommended, if external or internal vibration is used, vibrate the concrete following AASHTO T126 and record the vibration frequency and time.

After consolidation, strike-off and wood-float the concrete surface. Clean any excess concrete from the top and sides of the forms to achieve a level surface. Finish with minimum manipulation necessary to achieve a finished, flat, and even surface. Immediately transfer the specimens to the cure room and leave undisturbed for 24 hrs. Loosen the tie-



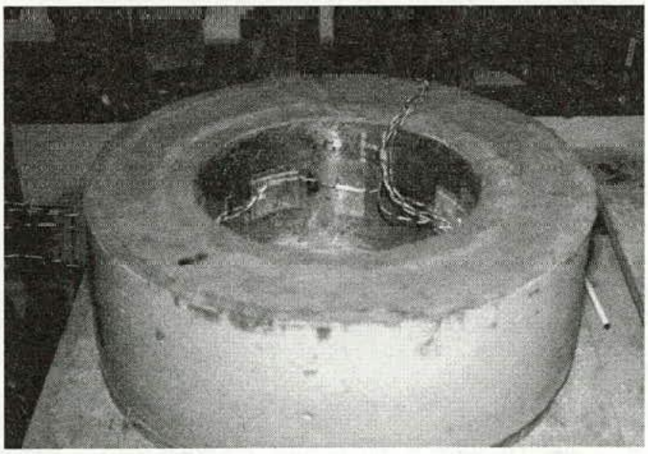



*Figure 1. Cracking-tendency test apparatus (left—empty, right—full of concrete).*

down holding the steel ring to the base form. Connect and begin monitoring the strain instrumentation immediately after placing the specimens in the cure room, cleaning the strain gage connecting wires with an emery cloth before attachment. After the concrete hardens sufficiently to resist indentation of the burlap, cover the specimens with wet burlap followed by plastic; keep the burlap wet until the forms are removed.

Unless otherwise required, remove the forms from the concrete rings at an age of 24 hrs $\pm$ 1 hr. Gently slide the ring or lift and tap the base to break the specimen free from the base form. Check that no debris is caught between the concrete and the base form. Keep the ring in contact with the base during testing or seal the bottom to prevent drying. Lightly dress the top edge of the concrete to remove sharp edges. Seal the top surface by running a bead of silicon caulk on the inside and outside edge of the top of the concrete ring and pressing a rubber mat or plastic into the caulk. Because some silicone caulks will corrode the steel ring, protect the top edge of the steel ring with a coat of varnish or do not allow the caulk to contact the steel.

### Testing

Monitor the strains in the rings as soon after casting as practical, recording strains every 30 minutes. Measure each strain gage separately. Periodically, review the strain measurements and visually inspect the ring for cracking. A large strain decrease in one or more gages indicates cracking. After cracking, note the cracking pattern and crack widths on the exterior radial face. Monitor the specimens for 2 additional weeks after cracking, measuring crack widths so the strain decrease and crack pattern can be characterized. Inspect the entire outside surface of the ring because a second crack occasionally develops on the opposite face of the ring from the original crack. If a second crack develops, monitor the new crack independently and note it in the final report.

### Calculation

Time-to-cracking is the age when strains measured by one or more of the strain gages mounted on the steel ring suddenly decrease. Average the results from each specimen cast from the batch, and report the age at cracking to $1/10$ of a day. If compressive strain increases in the steel ring are followed by gradual decreases and the concrete rings do not crack, report the results as "no cracking" and record the age when the test was terminated.

Plot the free-shrinkage strain of the unrestrained samples and determine the unrestrained shrinkage at the average time-to-cracking.

### Report

Record in the report the following data as pertinent to the variables studied:

- Properties of the concrete mixture: batch materials and proportions, air content, consistency, and unit weight of fresh concrete;
- Variations in ring dimensions, forming, casting, or curing;
- Steel ring thickness and outside diameter;
- Casting and curing temperatures;
- Temperature, relative humidity, and evaporation rate of the test room;
- Time-to-cracking in days for each specimen, and the average to $1/10$ of a day
- Average strain of steel ring at cracking;
- Plots of steel-ring strain versus time;
- Average unrestrained free-shrinkage at the average time-to-cracking; and
- Pattern of the cracking and the measured crack widths on the exterior face.

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Abbreviations used without definitions in TRB publications:

| | |
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| AASHO | American Association of State Highway Officials |
| AASHTO | American Association of State Highway and Transportation Officials |
| ASCE | American Society of Civil Engineers |
| ASME | American Society of Mechanical Engineers |
| ASTM | American Society for Testing and Materials |
| FAA | Federal Aviation Administration |
| FHWA | Federal Highway Administration |
| FRA | Federal Railroad Administration |
| FTA | Federal Transit Administration |
| IEEE | Institute of Electrical and Electronics Engineers |
| ITE | Institute of Transportation Engineers |
| NCHRP | National Cooperative Highway Research Program |
| NCTRP | National Cooperative Transit Research and Development Program |
| NHTSA | National Highway Traffic Safety Administration |
| SAE | Society of Automotive Engineers |
| TCRP | Transit Cooperative Research Program |
| TRB | Transportation Research Board |
| U.S.DOT | United States Department of Transportation |

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National Research Council
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Washington, D.C. 20418

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